PROCEEDINGS OF THE SEVENTH DREDGING SEMINAR

Prepared By

CENTER FOR DREDGING STUDIES
J. B. Herbich, Ph.D., P.E., Director

TAMU-SG-76-105
Report No. CDS-181
September 1975

TEXAS A&M UNIVERSITY SEA GRANT COLLEGE

PROCEEDINGS

OF THE

SEVENTH DREDGING SEMINAR

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Report No. CDS-181

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SEVENTH DREDGING SEMINAR
TULANE ROOM, BRANIFF PLACE HOTEL, NEW ORLEANS, LOUISIANA
NOVEMBER 8, 1974

Morning Session		*Moderator: Dr. John B. Herbich*
8:00 - 8:30	*Registration*	Braniff Place Hotel Lobby
8:30 am	*Welcoming Address*	Dr. John B. Herbich, Center for Dredging Studies, Texas A&M University.
8:50 am		"Use of Remote Sensing on Dredging Projects", Dr. Jerry Machemehl, Department of Civil Engineering, North Carolina State University at Raleigh.
9:25 am		"Overview of Dredged Material Research Program (DMRP)", Capt. R. M. Meccia, Coordinator for DMRP, Waterways Experiment Station, U.S. Army Engineers, Vicksburg, Mississippi.
	Break	
10:15 am		"Parameter Study of Variables Affecting the Performance of Hydraulic Pipeline Dredge Model", Dr. David R. Basco, Coastal, Hydraulic and Ocean Engineering Group, Texas A&M University.
11:05 am		"Maintenance Dredging in the New Orleans District, U.S. Army Corps of Engineers", Mr. H.R. Vick, New Orleans District, U.S. Army Corps of Engineers.
11:35 am		"Use of Catamaran Hulls for Sea-Going Cutterhead Dredges", Dr. John B. Herbich, Center for Dredging Studies, Texas A&M University.
12:15 pm	*Luncheon*	"The Port of New Orleans", Mr. Carl B. Hakenjos, Williams, McWilliams Co., Inc., New Orleans, presiding. Speaker: Colonel Herbert R. Haar, Jr., Associate Port Director, Port of New Orleans.
Afternoon Session		*Moderator: Dr. David R. Basco*
1:30 pm		"Salvops - Hopper Dredge Mackenzie", Colonel Don S. McCoy, District Engineer, U.S. Army Engineer District, Galveston, Texas.

2:10 pm		"Problems Associated with Submarine Pipeline Construction", Dr. Wayne A. Dunlap, Geotechnical Group, Texas A&M University.
2:40 pm		"Compressibility and Strength of Compacted Dredging", Max W. Giger and Dr. Raymond J. Krizek, Northwestern, Evanston, Illinois.
		"Permeability and Drainage Characteristics of Dredging", Dr. Raymond J. Krizek, Jan S. Jin, Abdelsalam M. Salem, Evanston, Illinois.
	Break	
3:40 pm		"Concepts for the Reclamation of Dredged Material", Charles W. Mallory, Hittman Associates, Inc. Columbia, Maryland and Capt. R.M. Meccia, Waterways Experiment Station, U.S. Army Engineers, Vicksburg, Mississippi.
4:20 pm		"The Effect of Suspended and Deposited Sediments on Estuary Organisms", Dr. Joseph M. O'Connor, Quirk, Lawer and Matusky, Engineers, Tappan, New York.
5:00 pm		"The Wonder of Water", a movie, American Water Operators.
5:30 pm		*Discussion and Announcements*
6:00 pm	*Adjournment*	

SPEAKERS
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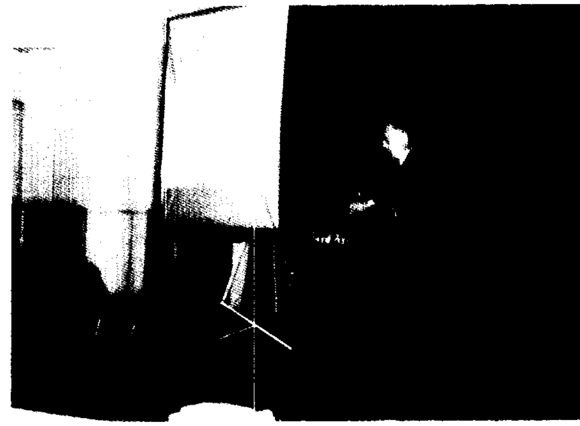


Col. Don S. McCoy

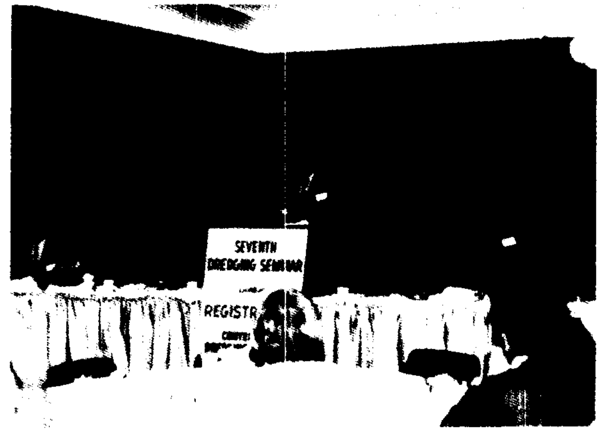
Dr. Jerry Machemehl

Cpt. Robert M. Meccia

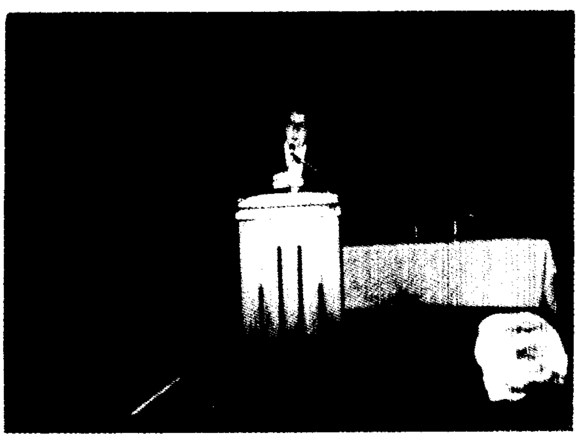
Dr. David R. Basco

Mr. Herman R. Vick

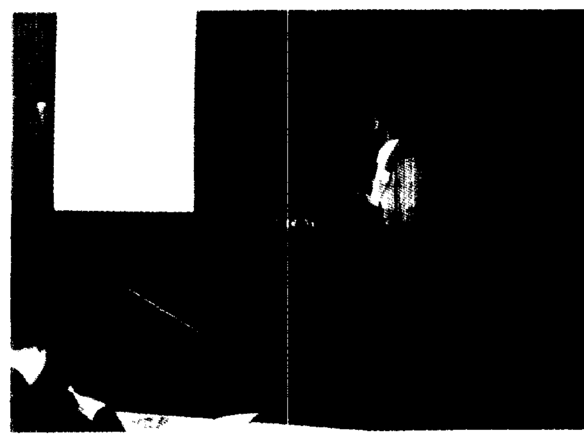
Dr. John B. Herbich



Col. Herbert R. Haar

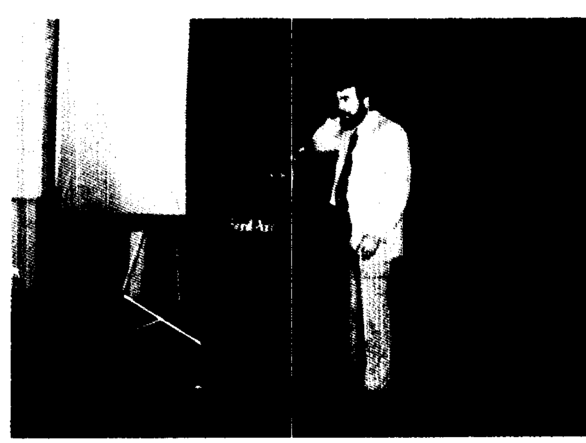


Dr. Wayne Dunlap



Mr. Max W. Giger

Mr. Charles W. Mallory



Dr. Joseph M. O'Connor

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ACKNOWLEDGMENT

The Seminar was arranged by Dr. John B. Herbich as part of the continuing educational activity of the Center for Dredging Studies.

The Proceedings were assembled and edited by Dr. Herbich. Editorial assistance of Dr. Gisela Mahoney was greatly appreciated.

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Mrs. Mattie Read and Mrs. Vicki Jamison typed the manuscript for publication. Mr. Bradford Hubbard, graduate student in ocean engineering, assisted in preparing the manuscript for publication.

SALVOPS - HOPPER DREDGE "A. MACKENZIE"

By

Colonel Don S. McCoy

District Engineer
Galveston District, Corps of Engineers
Galveston, Texas

MACKENZIE HISTORY

The U. S. Hopper Dredge A. MACKENZIE, until recently, was the oldest dredge in the Corps of Engineers fleet (Figure 1). She was launched in 1924 at Chester, Pennsylvania, and named for Major General Alexander Mackenzie, former Chief of Engineers, whose broad river and harbor engineering experience included development of the upper Mississippi River.

The A. MACKENZIE was a sea-going hopper dredge, and her 50 years of service extended from U. S. coasts to the South Pacific and back.

The MACKENZIE did her part to win World War II, and took her share of Japanese fire. She had steamed out of San Francisco for Midway Island in August 1943. On November 1, 1943, she was fired on for the first time, but it was nature that caused the most trouble.

She rode out her first typhoon at Okinawa during September 1945, but the following month - October, another typhoon with 147-mile-per-hour winds, crippled the MACKENZIE. A drifting vessel severed one of her anchor chains and the MACKENZIE was blown onto a sunken crane and beached. She was patched up and towed to San Francisco for repairs.

The MACKENZIE was headed for the scrap heap in 1951 when she was diverted to Galveston to do emergency dredging. She spent the last 23 years helping to keep the Gulf Coast ports open for world trade. For many years she was the only sea-going hopper dredge assigned to the Galveston District.

In 1967 she was joined by the dredge McFarland, the newest of the Corps of Engineers' hopper dredges.

The MACKENZIE continued to do her job, despite her years, and on March 6, we recognized 50 years of service by the durable old dredge. It was estimated that during her 50 years the MACKENZIE dredged 290 million cubic yards of silt -- a volume which if stacked on the Houston Astrodome would stand 1,900 stories in the air.

Fig. 1. U.S. Hopper Dredge A. MACKENZIE -- 1924-1974.

After the cermony she headed out again and began maintenance work on the Galveston entrance channel.

MACKENZIE COLLISION/SINKING

At 1:41 p.m., on April 24, while dredging the north half of the Galveston entrance channel, the U.S. Hopper Dredge "A. MACKENZIE" was involved in a three-vessel collision with the Norwegian Tanker "Bow Elm" and the University of Texas research vessel "Ida Green". She sank in 42 feet of water. All hands escaped without any major injuries, but the dredge was sitting on the bottom within 15 minutes after the collision (Figure 2).

As one of the crew members of the MACKENZIE put it: "We can all be thankful that the "Bow Elm" was empty." The "Bow Elm", a petrochemical tanker, received relatively minor damage to the bow.

The "Ida Green", a research vessel belonging to the Biomedical Research Institute of the University of Texas Medical Branch in Galveston, was the third vessel involved in the collision. The "Ida Green" also received relatively minor damage to the bow.

The impact of the collision and the tidal current in the channel, which is known to exceed five knots at times, caused the MACKENZIE to turn crosswise in the channel as she sank, blocking the north half of the entrance channel serving the ports of Galveston, Texas City and Houston.

CHANNEL KEPT OPEN

In order to provide uninterrupted use of the channel, we agreed to establish a bypass channel through the anchorage area, north of the site where the MACKENZIE sank, and north of the entrance channel.

The sunken dredge was marked by buoys and no further anchoring of vessels in the immediate area was permitted. Traffic through the bypass was limited to vessels drawing 36 feet or less, while vessels with a draft of up to 39 feet were allowed to use the south half of the channel, past the sunken dredge, during daylight hours (Figure 3).

During this period meetings were held with representatives of the Coast Guard, Galveston and Houston pilots' associations, and port directors, to apprise the various interests on the operation, special problems, and upcoming operations. Without full cooperation of each of the individual organizations, another major collision could have resulted. In addition,

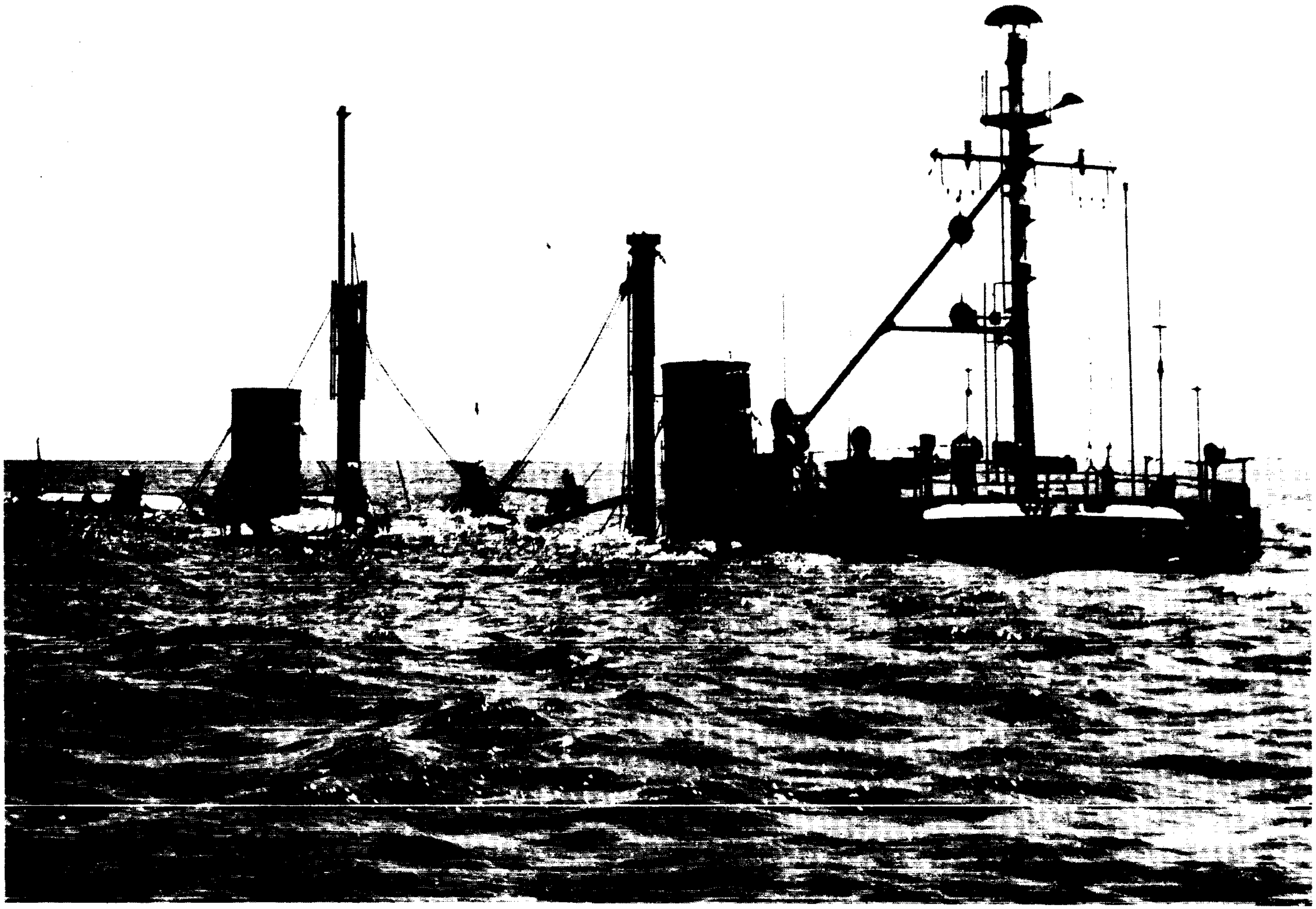

Fig. 2. Dredge A. MACKENZIE in Galveston Channel, April 24, 1974.

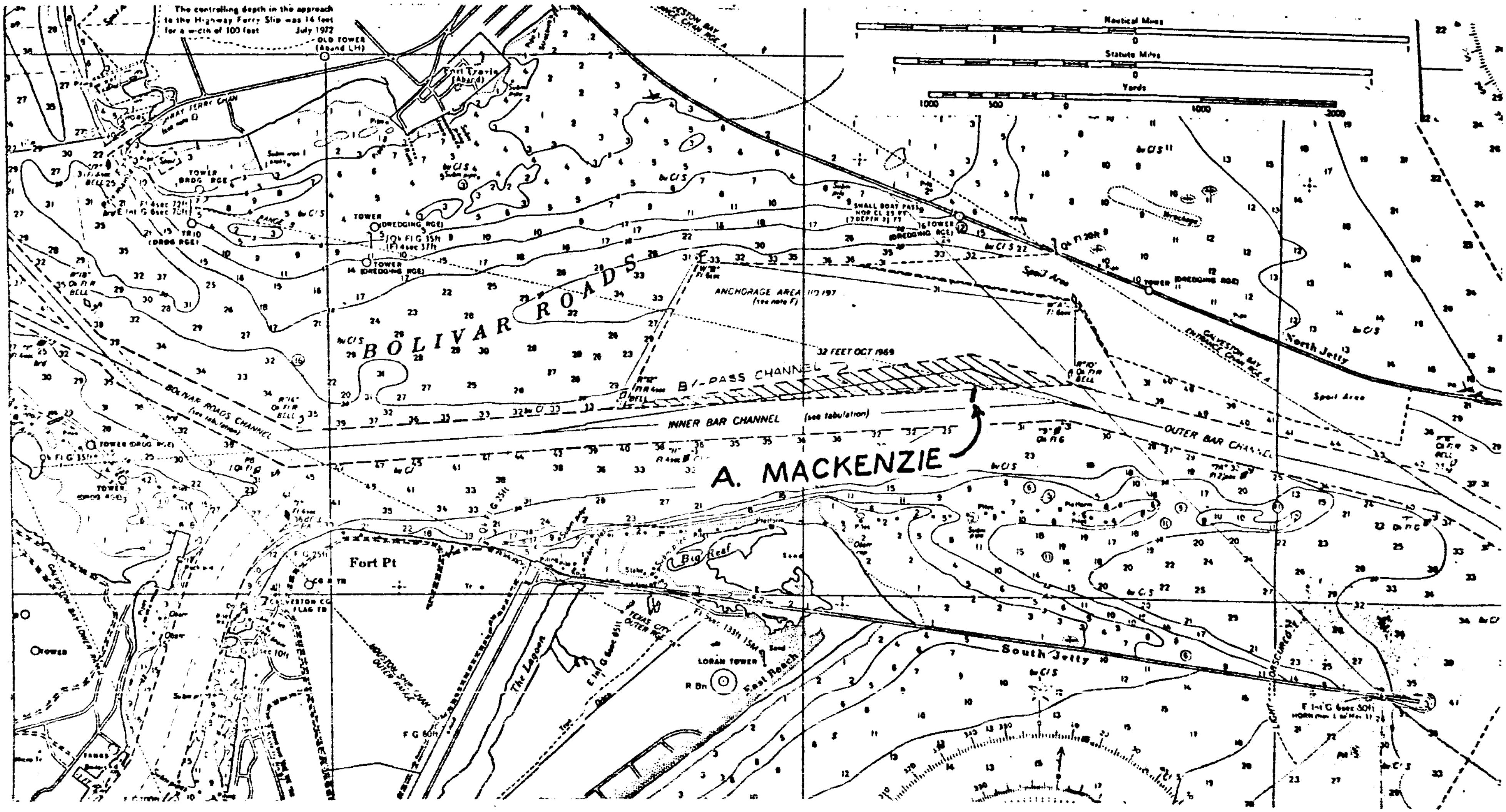

Plate — Locality Map

Figure 3. Location of the sunk dredge.

to these meetings, notices to navigation interests were issued in order to keep traffic flowing as smoothly as possible, without any undue delays.

Continued navigation in the entrance channel obviously was one concern. Another was getting the dredge out of the channel.

SALVAGE PLANNING

On April 25, less than 24 hours after the collision, we had hired a diving corporation from Pasadena, Texas, to make an inspection and survey damage to the sunken dredge. The recovery alternatives had to be examined and a decision made without delay.

The divers found that the MACKENZIE sank in a nearly upright position, almost perpendicular to the channel, with her bow against the North side of the channel.

The investigation revealed a large, jagged hole in the starboard side of the after-engine room, extending from the boat deck to about two feet above the bilge keel. The wooden boat deck was badly splintered, and the main deck jagged and pierced inward about six feet. Further inspection revealed no other damage around the ship. Soundings of the dredge hoppers indicated they contained about 550 cubic yards of silt, approximately 700 tons, or about one-third hopper capacity.

The top of the pilot house was above water, with about 10 feet of water over the boat deck or 18 to 20 feet over the main deck. No significant settling was noted.

At the time of sinking, the MACKENZIE had several thousand gallons of diesel fuel aboard. An odor of diesel fuel indicated some leakage, believed to be from the tank vents, and divers secured boots over the vents. In addition, a lightweight floating boom was rigged around the ship on April 25 by Corps of Engineers personnel.

On April 25—one day after the collision--I gathered consultants to advise me on the quickest, safest and most economical procedures for removal of the MACKENZIE. This included marine salvage consultants from Alexandria, Va., representatives from the Office of the Chief of Engineers and the Philadelphia and Detroit Districts.

MANAGEMENT/TECHNICAL OPTIONS

The consultants' findings showed three management options open for salvaging the dredge; namely, in-house, turnkey by contract and Navy Supervisor of Salvage. The advantages and disadvantages of each approach depended upon the availability of trained personnel, relative costs, character of record keeping and necessity for coordinating with other Federal and State agencies (Figure 4).

Included under each of the management options were two primary options-salvage for repair or scrap-and six principal technical alternatives (Figure 5).

1. Burial-in-place - <u>scrap</u>
2. Sheet-pile cofferdam - <u>either</u>
3. Raise by external lift - <u>either</u>
4. Raise with deck-edge cofferdam - <u>either</u>
5. Raise by internal buoyancy - <u>either</u>
6. Cut and wreck in-place - <u>scrap</u>

Various factors of time, cost, safety, pollution control, maintenance of navigation, risk, and availability of appropriate equipment were evaluated for each alternative. Additionally, an option was considered for removing the ship for sinking in 120 feet of water to be used as a reef.

DREDGE DECLARED A LOSS

After thorough investigation of the ship's condition, estimates of further damage due to salt water corosion, and estimates of cost to repair the electrical, hydraulic and mechanical components, I recommended on April 30 that the MACKENZIE be considered a total loss. I also recommended the ship be cut in place and removed for sale as scrap.

Since the sunken dredge severely constrained normal navigation traffic, and created a potential for additional collisions, I recommended that the safest and most expeditious method was that offered by the Navy Supervisor of Salvage. On May 4, the Office of Chief of Engineers approved our plan to utilize the Supervisor of Salvage, U. S. Navy, to provide consultant services, and to act as prime contractor for selected salvage operations.

SALVAGE BEGINS

An agreement between the Galveston District and the Supervisor of Salvage was reached, and the salvage operation began. Our first step was to rent a barge, put a crane and an office on it, and make preparations for the divers to make a more detailed inspection.

MANAGEMENT OPTIONS

- IN-HOUSE
- TURN-KEY BY CONTRACT
- NAVY SALVAGE

Figure 4. Management Options.

TECHNICAL ALTERNATIVES

SALVAGE

SCRAP

- BURIAL-IN-PLACE - SCRAP
- SHEET-PILE COFFERDAM - EITHER
- RAISE BY EXTERNAL LIFT - EITHER
- RAISE WITH DECK-EDGE COFFERDAM - EITHER
- RAISE WITH INTERNAL BUOYANCY - EITHER
- CUT AND WRECK IN-PLACE - SCRAP

Figure 5. Technical Alternatives.

Two mobile office trailers were rented and set up at the Galveston District hopper dredge dock so that the Corps, Navy and contract personnel would be in an office close to the site.

The Supervisor of Navy Salvage obtained the services of Murphy-Pacific Marine Salvage Co., under existing salvage contracts. Key Navy personnel were mobilized, including a logistics analyst who coordinates use of Naval salvage equipment.

A marine architectural firm was selected to perform the analysis required to develop a final salvage plan. Also, a subcontract for diving services was executed, and on May 7, two days after the arrival of the first contractor representative, salvage operations were underway.

SALVAGE ORGANIZATION FORMED

Because of the complexities of this salvage operation, we established a separate task force and assigned a Resident Engineer to coordinate and supervise the various components of the effort. High priority was given to all activities relating to the salvage operations.

Whenever you are responsible for an unusual or non-standard operation, the first thing you have to do is determine who is going to do what, to whom, and who is responsible for what. After this is determined, that is determination of lines of authority and responsibility, you publish guidelines for all concerned, and insist on compliance with these procedures.

Figure 6 shows the Corps of Engineers' personnel, the Navy, the civilian contractors, and the Coast Guard. Solid lines are command lines, and dotted lines are coordination lines. This organization functioned effectively and efficiently.

CRITICAL PATH METHOD (CPM)

The next step is to make a CPM chart--critical path method analysis (Figure 7). Under this procedure, you determine what activities must be accomplished, in what order, and the estimated time required. Having done the analysis, you closely monitor actual accomplishment to insure the proper equipment and personnel are available when required, coordination of safety is timely, and wasted manpower and costs are minimal.

For instance, when we made the first lift, I had to insure the oil pollution gear was ready, the lift barges were there, the crane was on station

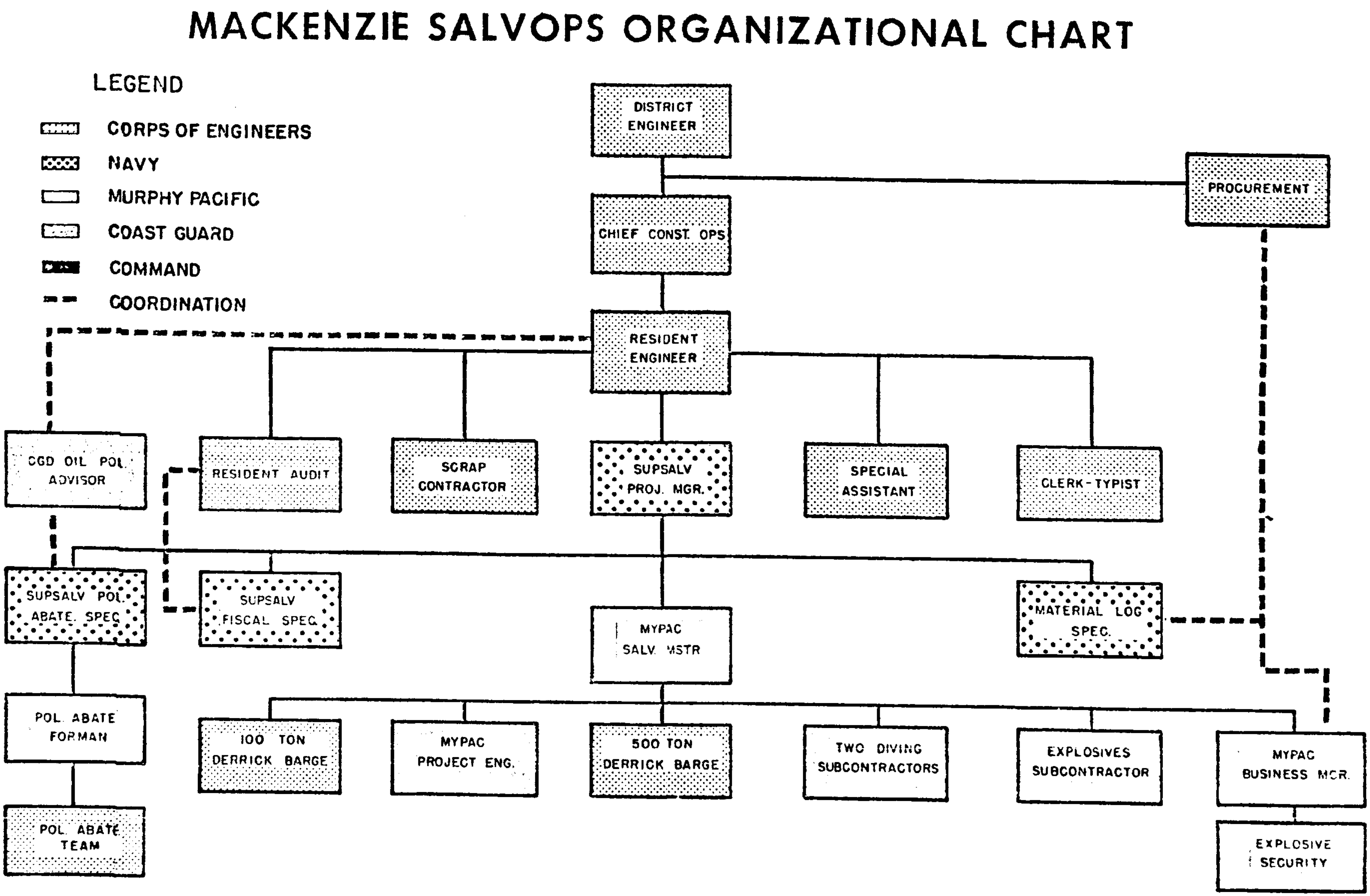

Figure 6. MACKENZIE Salvops Organizational Chart.

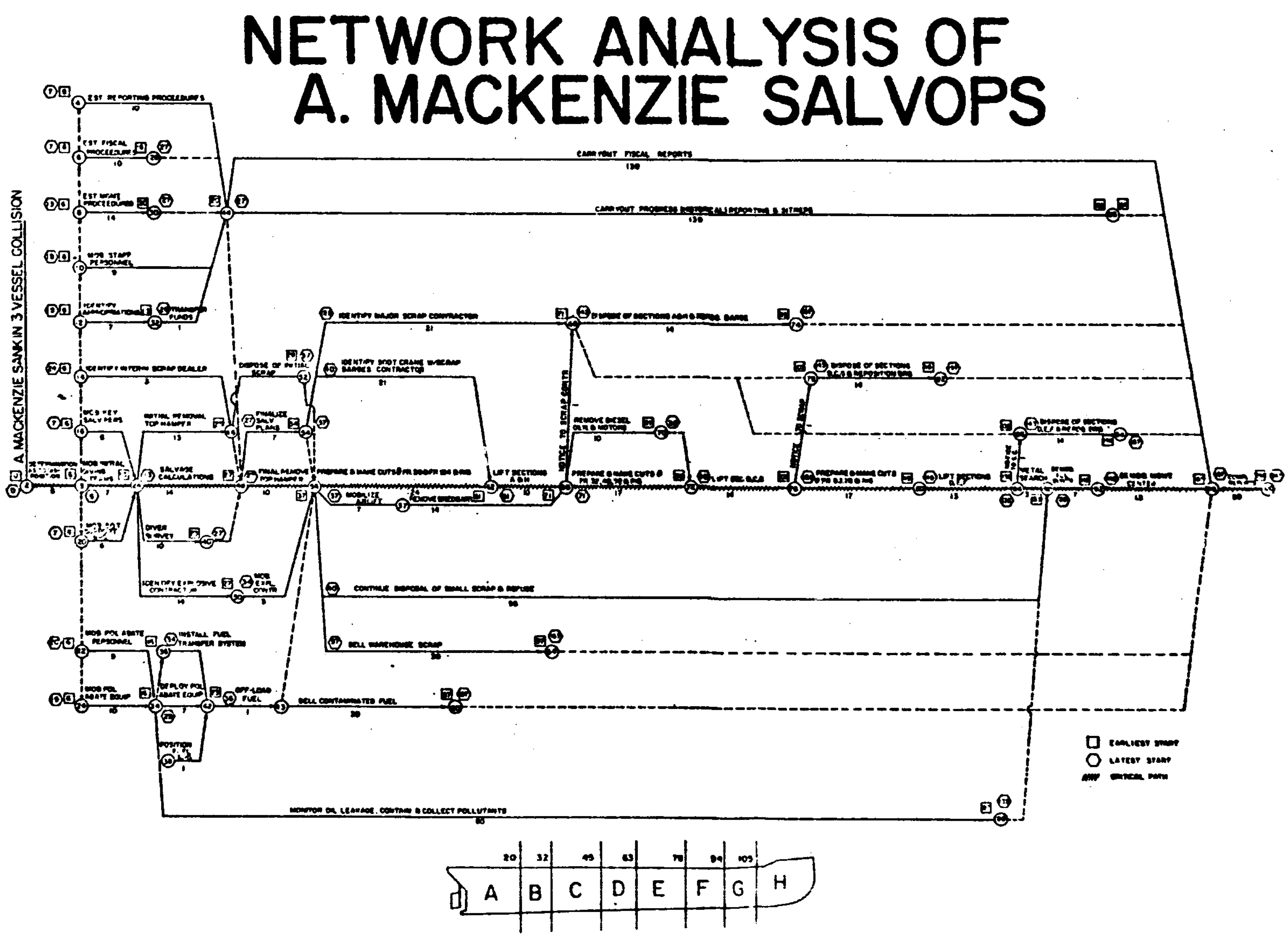

Figure 7. A Network Analysis of A. MACKENZIE Salvops

and the channel closing notices were issued. all these items are done by different people at different locations. This is the reason a CPM analysis and performance monitoring are useful.

For monitoring, the CPM was posted in the field office, in the District Office, and on the work barge. As work was completed, a completion date was posted on the CPM. If the date is red, we didn't make it on time, if it is green, we made the date or beat it. There is only one red mark on the critical path and it is for a three-day delay.

POLLUTION CONTROL

Shortly after the work barge was located at the site of the MACKENZIE, the oil containment boom was received from the various naval storage depots. A work force of personnel from the A. MACKENZIE assembled the containment boom by bolting together the floats, inflating, and towing the boom to the site utilizing Corps tugs (Figure 8).

The Navy oil skimmer, a vessel designed to gather the oil or other debris, arrived in Galveston on May 12 (Figure 9). Mooring anchors were installed around the MACKENZIE by the Coast Guard on May 14, and on May 17 the first tests were run.

Operations resumed the following morning, and by noon about 10,000 gallons of diesel fuel had been removed from the MACKENZIE and pumped aboard the fuel barge. Divers had installed hoses inside the ship's fuel tanks, pumped water into the tanks, forcing the oil out and into the barges.

The operation was completed by mid-afternoon with no oil spills.

TECHNICAL ALTERNATIVES

As the salvage operations progressed, detailed evaluations were made on the various plans for removing the ship. Of the six technical alternatives mentioned earlier, four were discarded.

Burial in place was unacceptable because there is a possibility that the channel will be deepened, and widened in the future.

A sheet-pile cofferdam was unacceptable due to the depth of water, exposure to the open seas and possibility of hurricane storms.

The option of raising by external lift was ruled out due to unavailability of lift equipment, and the high risk of failure.

Success or failure of a deck edge cofferdam would have depended

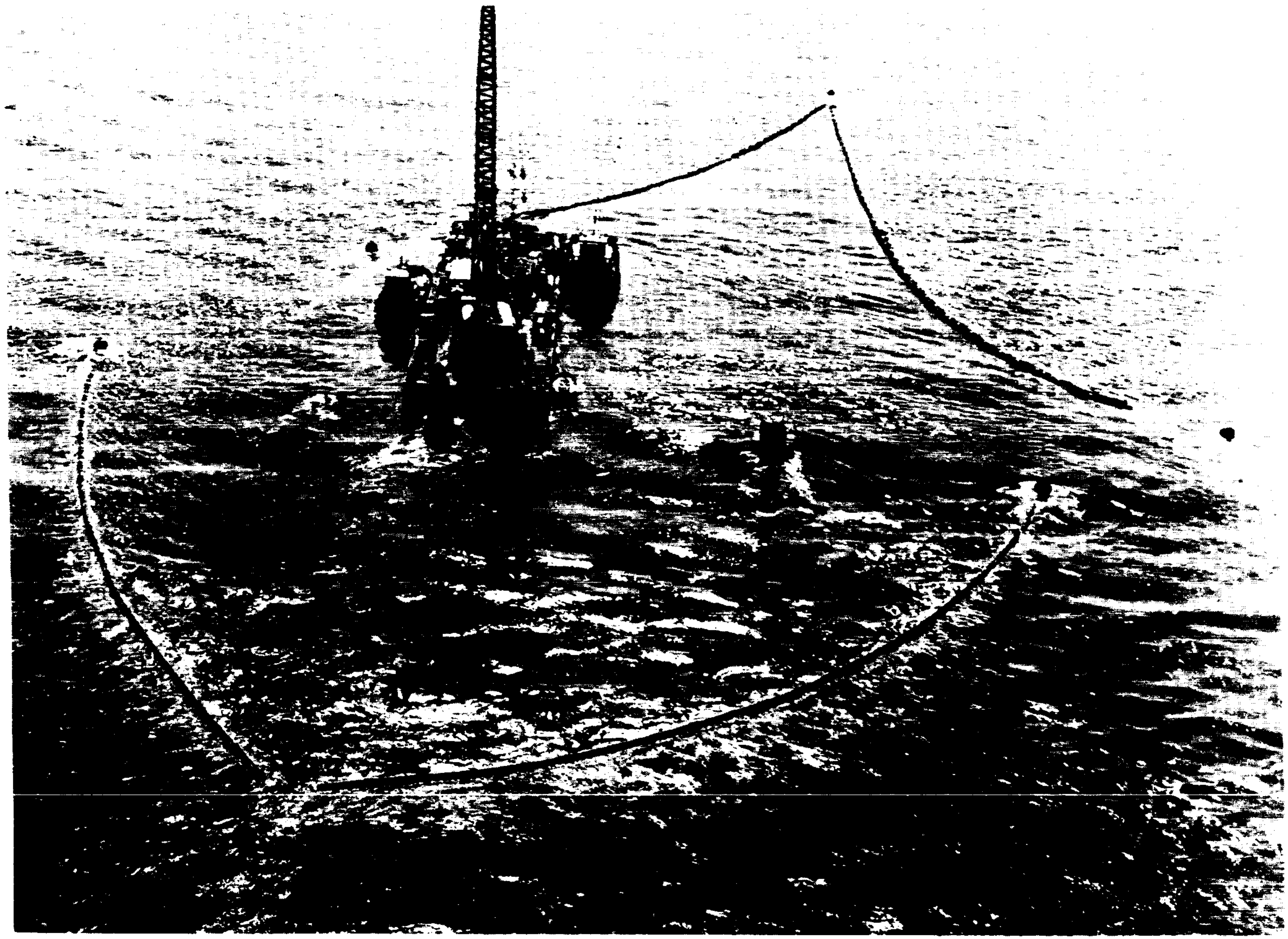

Fig. 8. Oil pollution boom surrounding salvage site. Ship's stacks seen standing above water.

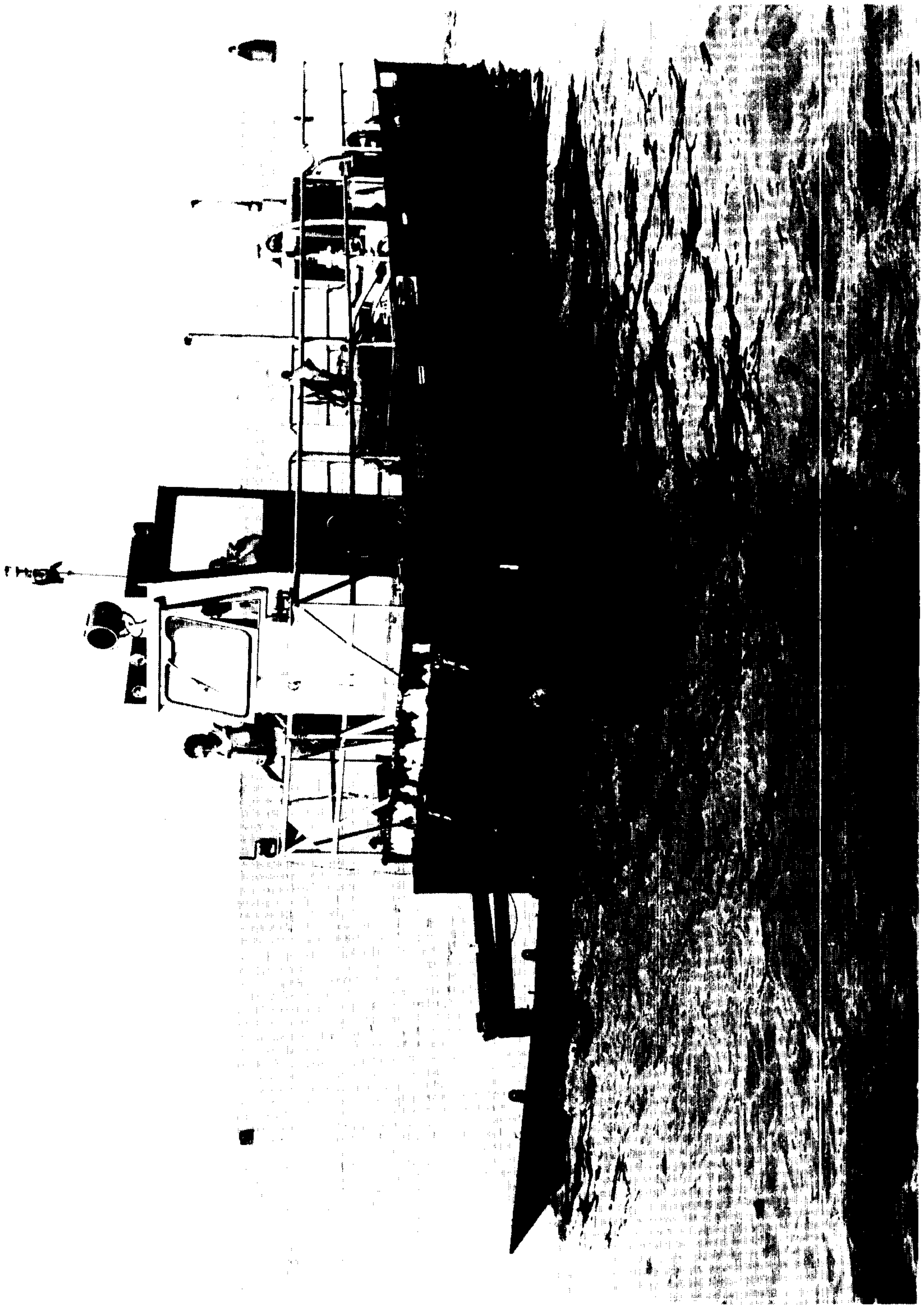

Fig. 9. U. S. Navy oil skimmer in Galveston Channel.

largely on the MACKENZIE staying in an upright, stable condition. She was sitting broadside to the current, and therefore the stability of the ship could not be assured.

Two plans were proposed as workable salvage plans.

Raising with internal buoyancy entailed cutting the vessel at two points, removing the hoppers in the center of the ship, and towing the two remaining sections to sea where they would be sunk with explosive charges. Internal buoyancy systems could not have lifted the entire ship because the hoppers were open on top and had no structural ability to support the buoyancy forces. Several methods of internal buoyancy were explored, of which the best was determined to be the use of foam. If a good quality foam could be obtained and placed, salvage consultants said, the two sections would float with boat deck awash. However, because the 120 foot depth curve is about-100 miles from Galveston, it was questionable if these two sections could be towed safely that far and if they could be sunk without scattering debris.

The final plan, and that approved for the salvage of the MACKENZIE, was to cut and wreck the dredge in place, and dispose of the scrap by sale to the highest bidder. Even though it was estimated that this plan would take about 30 days longer than the internal buoyancy method, risk of failure was minimized.

This plan involved removal of all materials and structures above the boat deck and vertically cutting the ship into at least eight sections. After the first lift sequence, section weights were re-analyzed and additional horizontal cuts were made.

SALVAGE OPERATIONS

I have discussed the analysis of alternatives, the general organization concepts, the CPM plan of action, and our early steps to provide protection from oil pollution. Now in more detail, I will cover the sequence of events leading to completion of the salvage operations.

After the work barge was on site, oil was removed from the MACKENZIE's fuel tanks, and additional structural and position inspections were made by divers. Workers removed material and structures above the boat deck, including the mast from the flying bridge, the drag tender's shack and drag tender's quarters. Divers then began the first traverse cut underwater, immediately ahead of the ship's bridge. The two updraft engine room stacks were partially cut away and platforms installed for diver access into the ship's interior. This procedure saved considerable time and expense by eliminating the necessity for fabricating an access cofferdam. As the salvage operation progressed, these stacks were moved to permit easier access to the ship and protection from the tidal currents.

EXPLOSIVES

A subcontract was awarded to an explosives firm from Harvey, Louisiana, to furnish technicians and explosives to assist in the cutting operation. The necessary permits were acquired to store and transport explosives to the wreck site, and portable magazines were set up and barricaded. Prior to the arrival of the explosives, meetings were held between the contractor, Coast Guard, and Corps of Engineers representatives to assure utmost safety. Due to the close proximity of inhabited buildings and radio transmitters to the dock, strict compliance with handling procedures was dictated and a 24-hour a day guard service was engaged to prevent theft of the explosives or caps.

Sheet metal shaped charges were designed and fabricated by the project engineer and explosives technician. The shaped charge concentrates the explosive force for effective line-cutting of metals. One set of forms held 2.2 pounds of C-4 explosives per linear foot, and the other held 0.8 pounds per linear foot. C-4 plastic explosive is well suited for this use since it is pliable, extremely stable, and has a high velocity rate of detonation.

The first shot was fired on June 1 on the main deck of the MACKENZIE, using 8.8 pounds of C-4 in one 48-inch shaped charge. This was essentially a test shot to evaluate the effects of the blasts on marine life in the area, and to determine the cutting efficiency and proper placement procedures.

The fish kill was negligible and the cutting action was greater than predicted. A number of hardhead catfish and two angel fish were disabled on the first shot, which the birds quickly picked up. Texas Parks and Wildlife personnel were on hand to assist in the evaluation of explosive procedures as related to protection of fish.

Following the shot, the oil skimmer was used to scoop up debris. Subsequent shots, however, convinced us that the explosives disarranged equipment inside the wreck to such an extent that the divers were hampered by the debris. We decided that explosives should be used only to supplement the oxy-arc cutting. Explosives were used, however, to clear the timber from the boat deck, as seen here, or to cut drain holes to facilitate drainage during lifting, and to make the final separation as each section was severed (Figure 10).

"AS-BUILT" MODIFIED

During the operation we found that our "as-built" drawings were not as accurate as required because of the numerous modifications which were made over the ship's 50-year life. We had to continually evaluate and re-evaluate.

When positioning a cut, we had to consider where we would find the least number of connections, to assure that there were no remaining structural connections when the lift was made. We were restricted by the crane lift capacity, and since the rigging had to be connected by hand, we were restricted by the size of rigging which a diver could place under water in heavy currents.

Lifting the materials and structures above the boat deck was handled by the 110-ton derrick on the work barge. However, all major lifts were performed with a 500-ton derrick barge. This ocean crane, used in the offshore oil tower construction along the Louisiana coast, was moored alongside the A. MACKENZIE on June 24.

FIRST MAJOR LIFT

Upon completion of the lift connections, hoisting commenced. It soon became obvious that complete separation had not been made. After work by the divers, the bow section of the MACKENZIE was lifted to water level and cleared of additional weight, including silt which was washed out to reduce the weight. The bow came up at 320 tons (Figure 11).

Next was the stern, which weighed 370 tons. The bow and stern were placed on a scrap barge, and secured. Investigations showed that the higher than predicted lift weights had resulted from two main causes--trapped water, and silt deposits greater than expected (Figure 12).

In view of this, we decided that five of the six remaining sections, would be cut longitudinally at about the 12-foot waterplane. In addition, all pieces, no matter what their estimated weights, would be rigged for the capacity of the crane (Figures 13 & 14).

SECOND AND THIRD MAJOR LIFTS

The divers returned to their cutting program and the second major lift began as scheduled on July 22, followed by the final lift which began on

Fig. 10. Colonel Don S. McCoy, second from left, explains use of shaped charges seen in foreground.

Fig. 11. Bow lift by 500-ton offshore crane.

Fig. 12. Bow and stern during first lift.

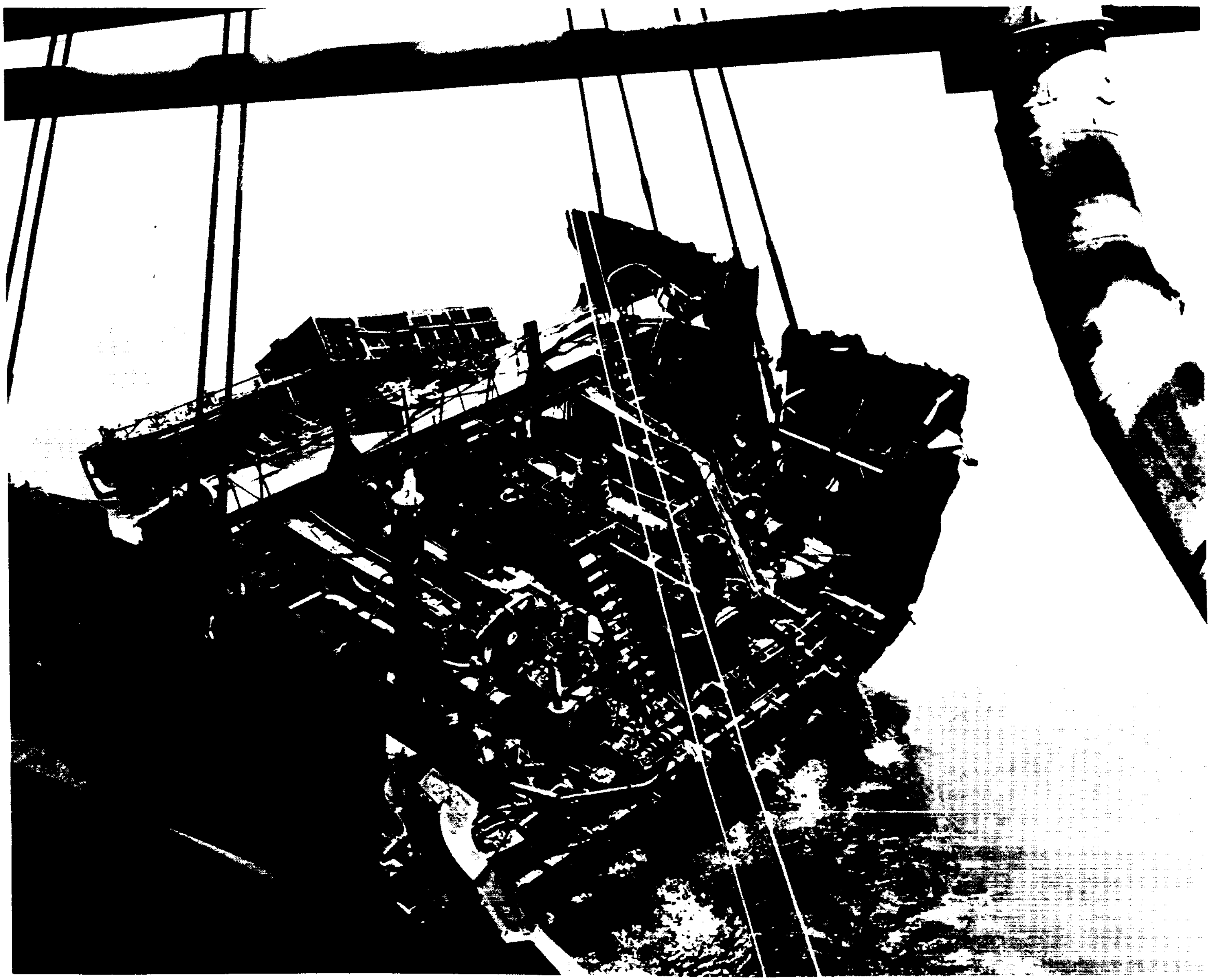

Fig. 13. Section of MACKENZIE's engine room during final lift, revealing hole in ship's side, from collision, seen at right.

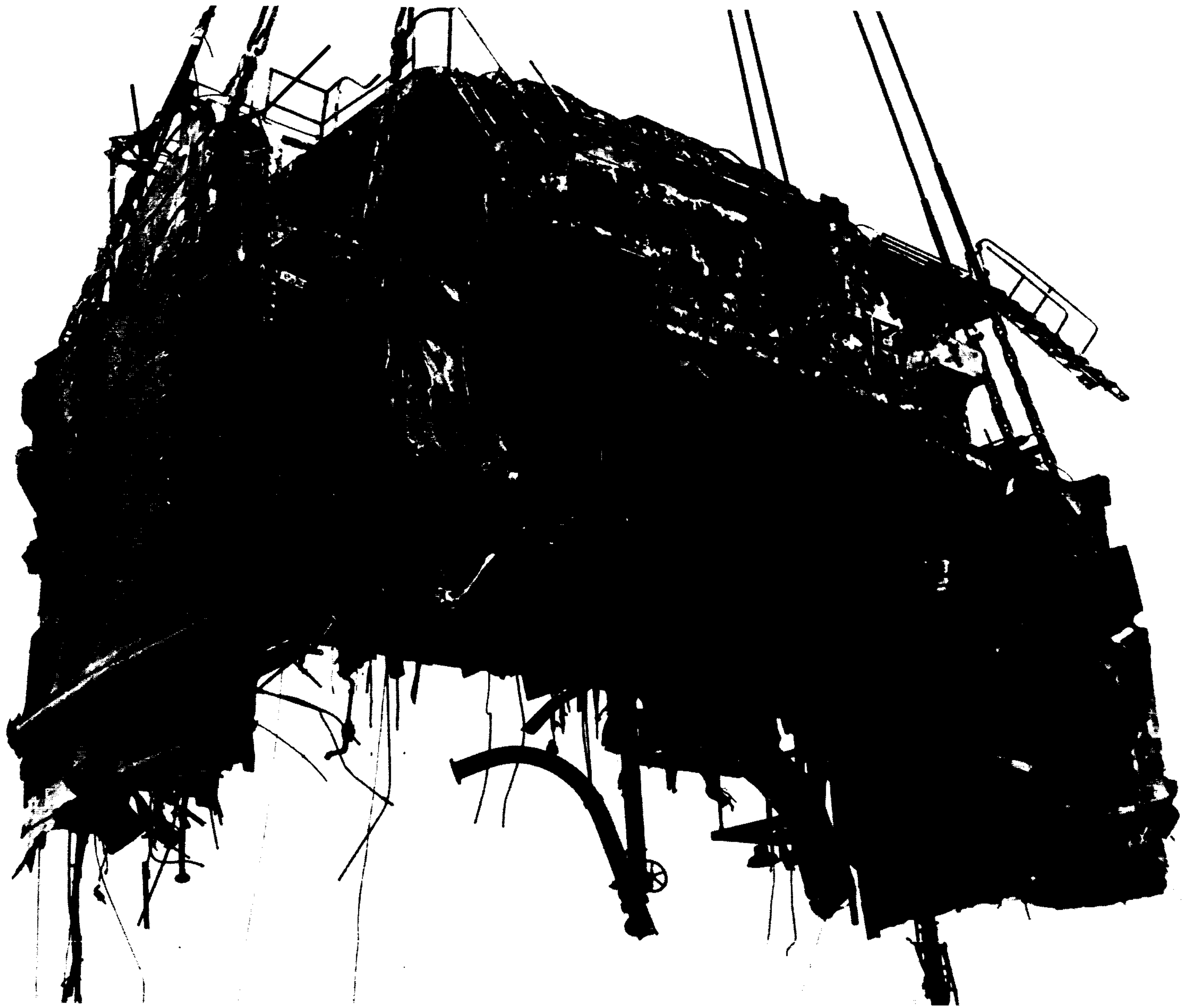

Fig. 14. Portion of ship's mid-section during second lift.

August 19. By 1:15 p.m., on August 20, the last section was placed on the scrap barge.

The derrick barge had retrieved her anchors and cleared the wreck site by 5 p.m., completing removal of the main sections of the MACKENZIE. Final stage of the operation was the metal search conducted by divers who laid out the bottom area in a grid and physically "walked" the channel bottom, removing all scraps left from the operation.

Throughout the operation we were constantly subjected to passage of large ships, which passed within 50 to 100 feet of the work barge. However, with completion of the metal search, on August 30, we were able to move the work barge out, and the Galveston District's Hopper Dredge McFarland started dredging silt from the area to remove localized shoals.

By September 16, a channel depth of 40 feet was attained and at 4 p.m. on that date the Coast Guard placed the channel buoys back in their original positions and the channel was restored to normal operation.

SUMMARY

The MACKENZIE salvage operation was completed in 117 working days--13 days ahead of schedule. The job was accomplished without any shipping mishaps despite the close quarters between ship traffic to and from the ports of Galveston, Texas City, and Houston. An estimated 4,700 ships used the channel during this period, with only brief delays, if any, caused by the salvage operation. There were no injuries incurred by the work crew which numbered up to a peak of 125 persons during the lifting operations.

Environmentally, the entire operation was accomplished with only relatively minor fish loss, and this occurred essentially only at the time of the first use of explosives. Elaborate preparations were made to control any oil spillage which might have occurred, but there was virtually no oil loss from the MACKENZIE, and there was no known environmental damage.

USE OF AERIAL REMOTE SENSORS TO MONITOR DREDGING PROJECTS

By

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INTRODUCTION

The application of aerial remote sensing to dredging projects has been neglected in the body of literature.[3] The primary objective of this paper is to briefly review the application of aerial remote sensing to the dredging industry.

AERIAL REMOTE SENSORS

The primary aerial remote sensors considered in this paper are black and white, black and white infrared, color, and color infrared aerial photographs and multispectral aerial photographs. Each of the aerial remote sensors has a characteristic particularly useful in planning, designing and/or controlling a dredging project.

Black and White Aerial Photographs - The conventional black and white (panchromatic film) aerial photograph produces images consisting of shades of gray (varying from completely white to completely black). The panchromatic film is sensitive to electromagnetic energy within the visible portion of the spectrum from about 400 to 700 millimicrons.

As reported by Stafford[3], the black and white aerial photograph has been used extensively in coastal studies because of its general availability, relatively low cost and versatility. The black and white aerial photograph provides an overall view of an area that cannot be obtained from ground observations.

The black and white aerial photograph would be particularly useful to the dredging industry as a map/graphic display of a dredging project. A series of photographs taken during a dredging operation could then be used to document the project.

Black and White Infrared Aerial Photographs - The black and white infrared aerial photographs produce tones of gray on the positive print.

The range of spectral sensitivity of the film is extended beyond the visible band into the near-infrared wavelength band. The film is sensitive to wavelengths between 400 and 900 millimicrons.

The black and white infrared film produces a sharp contrast between different reflecting surfaces and is particularly useful in delineating/locating the shoreline/waterline in an area. The black and white infrared film also has an increased haze penetration capability since the film is exposed to relatively long wavelengths.

The black and white infrared photograph would be particularly useful to the dredging industry in delineating the shore/water interface at different stages of tide.

Color Aerial Photographs - The color aerial photograph produces images in color. The film is typically composed of three emulsion layers, each of which is sensitive to wavelengths corresponding to a particular color. Emulsion layers sensitive to the blue, green and red bands are normally used. A chemical reaction occurs when the film is exposed to light. The dyes in the emulsion layers combine to produce colors corresponding to the color of the object being photographed.

The addition of color to the aerial photograph increases the detection and discrimination capability. The three variables of hue, brightness and saturation add to the capability to differentiate between objects.

As reported by Stafford[3], the color aerial photograph has been used extensively in coastal studies to distinguish and identify objects. The color aerial photograph is particularly useful in investigating underwater phenomena. Berryhill[1] found color aerial photographs best for detecting water current patterns and for observing bottom topography. Color aerial photographs have also been found best for water movement and pollution studies. In recent years color aerial photographs have been used in water depths determination and water penetration studies. The maximum depth penetration has been found to vary with the amount of suspended sediment and organic particulate matter in the water.

The color aerial photographs would be particularly useful to the dredging industry in investigating water current patterns, sediment patterns, shoaling areas and water depths at the dredging site.

Color Infrared Photographs - Color infrared aerial photographs are different from conventional color photographs in that the emulsion layer sensitive to the blue band is replaced by an emulsion layer sensitive to the near infrared band. The presence of the emulsion layer sensitive to the near infrared band extends the spectral sensitivity of the film from about 700 to 900 millimicrons. The addition of the emulsion layer sensitive to the near infrared band increases the detection and discrimination capability. The false colors of the photograph add to the capability to differentiate between objects.

As reported by Stafford[3] the color aerial photograph has been used extensively in investigations of the coastal environment. Stroud and Cooper[4] found color infrared aerial photographs superior to color photographs in their investigation of marsh vegetation and marsh productivity.

The color infrared aerial photograph would be particularly useful in monitoring the ecology of a project area.

Multispectral Aerial Photographs - Multispectral aerial photographs (produced with multiple lens cameras with different films and/or filter combinations) with various image-enhancing techniques have increased the capability of detecting water depth. Yost and Wenderoth[5] used multispectral aerial photographs in 1968 to investigate the exposure and spectral filtration conditions that would maximize water penetration. A four-lens multispectral camera with filters was used to produce multispectral photographs exposed with light within a particular wavelengths band. Yost and Wenderoth[5] reported that the green spectral band had twice the water penetration capability of the red band and three times that of the blue band.

Ross[2] used multispectral aerial photographs in 1969 to investigate the water penetration characteristics of different bands of the visible spectrum. A four-lens multispectra camera was used. Ross[2] reported that two spectral bands, 460-510 millimicrons and 510-560 millimicrons, should be used in water depth penetration (if one band is used Ross recommended the 460-580 millimicron band). He emphasized the usefulness of the blue wavelength in the band in depth-penetration studies.

Yost and Wenderoth[6] employed a four-lens camera to produce multispectral photographs in four bands ranging from 360 to 900 millimicrons.

A multispectral additive color-viewing system was employed to form a single color/false color presentation. Maximum water penetration for clear water was obtained with a wavelength of 480 millimicrons. In an investigation of the factors which affect the ability to detect and identify underwater objects, Yost and Wenderoth[6] found that the use of multispectral aerial photographs and the additive color-viewing technique provided results that were superior to color aerial photographs. The development of techniques to use multispectral aerial photographs and additive color viewing to determine shallow water depths has a future in the dredging industry.

Multispectral aerial photographs can be used in the planning, designing and/or controlling of a dredging project. In the planning and designing of a dredging project, one film and/or filter combination (wavelength band) could be chosen to give maximum water penetration, while another film and/or filter combination (wavelength band) could be chosen to discriminate between vegetation species, etc.

APPLICATION OF AERIAL REMOTE SENSORS

Aerial remote sensors can be utilized to obtain data shown below:

I. Coastal Zone

- a. Coastline/shoreline/interface
- b. Terrain/landform/drainage
- c. Erosion
- d. Resources/vegetation

II. Nearshore or Estuarine Zone

- a. Waves surfaces
- b. Wave patterns
    - (1) Refraction
    - (2) Diffraction
- c. Water
    - (1) Currents
    - (2) Depth/penetration/bottom topography
- d. Sediment patterns
- e. Environmental
    - (1) Pollution/turbid plumes
    - (2) Thermal anomalies/effluent diffusion

CONCLUSION

Aerial remote sensors can produce an economic savings and/or improvements in the quality and quantity of data collected for designing, planning and controlling a dredging project.

NOTE

The oral presentation of this paper was accompanied by 35 mm slides in black and white, and in color. Because of the loss of quality that would result in the printing of color illustrations in a black and white format, it was decided to delete the illustrations from the paper.

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DREDGED MATERIAL DISPOSAL EFFECTS AND ALTERNATIVES

By

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Dredging to create and maintain navigable channels for the Nation's waterborne commerce -- a basic activity and responsibility of the Corps -- inherently requires removal of large quantities of sediment, which must be disposed of economically but with the least possible adverse environmental impact. But just what are the impacts or effects, and what are the alternatives?

Localized studies have been made to investigate the environmental impact of specific disposal practices and to explore alternative disposal methods; however, these have not provided sufficient, definitive information for general application or predictive capabilities. Therefore, a research program of national scope is underway at the Corps' Waterways Experiment Station (WES) to seek answers to basic and critical questions.

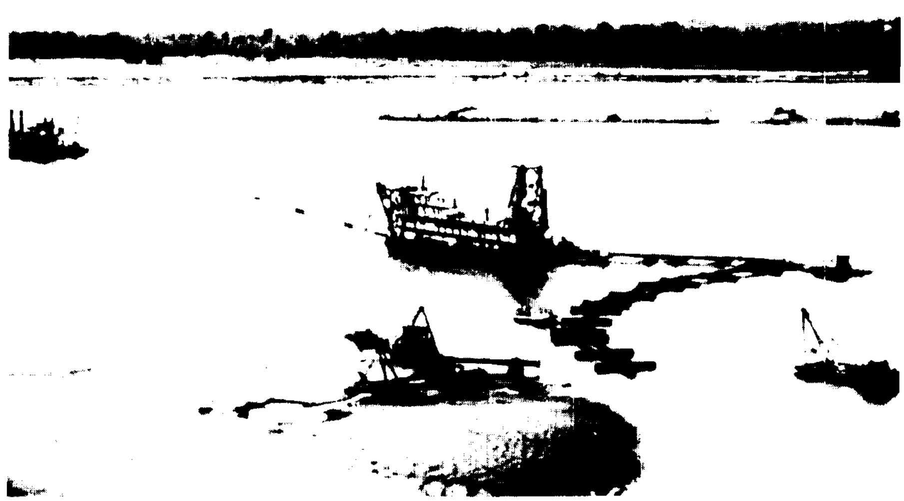

Fig. 1. Millions of cubic yards of sediment must be dredged annually to maintain navigation channel depths because of the effects of shoaling

The answers will come from the combined field and laboratory investigations being conducted at WES within the Environmental Effects Laboratory (EEL). The objective of the Dredged Material Research Program (DMRP) is to develop technically satisfactory, environmentally campatible, and economically feasible alternatives for dredging and disposal. This includes considering the dredged material itself as a manageable resource, whereas dredged material had once been regarded as a nuisance waste product to be dumped in open waters or on marshes without any awareness of the environmental consequences.

Lack of knowledge about these consequences has caused the use of interim measures in an effort to reduce or prevent degradation of water quality. These measures include confining dredged material within dikes and attempting to analyze the sediments themselves for traces of pollution. Since the effect of dredging on water quality has been acknowledged as an unknown more than a reality, Congress accepted the need for fundamental and applied research to provide permanent solutions.

Congress authorized such research in Section 123(i) of the River and Harbor Act of 1970, and eventually WES was assigned the challenge. In February 1973 (after a year and 8 months of problem identification and assessment, and research program development), the Office of Management and Budget approved a 5-year, $30 million program with the understanding that any research efforts would be cut off if they did not give immediate promise of effective results for the Nation.

THE FOUR PROJECTS

The first DMRP annual report, published March 1974, illustrates initial accomplishments in defining the problems and outlining the goals established for each of the seven original areas of research. Since then, program goals and tasks have been consolidated into four areas directed by project managers responsible for:

Aquatic Disposal
Habitat Development
Disposal Operations
Productive Uses

This structure makes it easier to manage and coordinate the entire program while disseminating the results of each project to Corps Districts and other agencies without having to wait for the entire research program to be completed. Thus, immediate benefits from the ongoing research may be used in assessing the environmental impacts of disposal operations and also in planning and implementing future dredging and disposal projects.

DREDGED MATERIAL RESEARCH PROGRAM			
Aquatic Disposal Research Project	**Habitat Development Research Project**	**Disposal Operations Research Project**	**Productive Uses Project**
Coastal Disposal Area Field Research	Upland and Marsh Disposal Environmental Impacts	Containment Area Operation Research	Aquatic Disposal Concepts Development
Movements of Dredged Material	Artificial Marsh and Island Creation	Dredged Material Densification	Upland Disposal Concepts Development
Effects of Dredging and Disposal on Water Quality	Habitat Development Research	Treatment of Contaminated Dredged Material	Land Improvement Research
Effects of Dredging and Disposal on Aquatic Organisms		Turbidity Prediction and Control	Products Research
Pollution Status of Dredged Material		Basic Equipment Related Studies	Disposal Area Land Use Concepts

Table 1. Dredged Material Research Program

Aquatic Disposal

The largest of the four research projects is concerned with aquatic disposal. Research undertaken within this project involves multiple aspects of the effects of open-water disposal -- the traditional method (confining dredged material on land did not begin until a couple of decades ago). Although open-water disposal is the least expensive from the dredger's point of view, researching its effects on the marine environment is the most expensive part of the DMRP.

In years past all dredged material was considered the waste byproduct of a dredging operation. The dredging itself was the major concern because it accomplished a recognized good -- the initial development and continued maintenance required of a channel used for navigation. The upswell of environmental awareness that has taken place in recent years focused attention on this method of disposal. After all, dredged material constitutes the largest quantity of matter placed or redistributed by man in the marine environment. Although long-term damage to ecosystems has been documented in only a few instances, the fear of adverse consequences has forced us to expend tens of millions of dollars for the widespread precaution of putting dredged material on land and confining it.

The question we are attempting to answer is: Under what conditions will the magnitude of environmental impact of openwater disposal by sufficiently adverse to warrant expenditure of significantly larger amounts of money for alternative disposal methods? A basic tradeoff is involved here.

Fig. 2. Hydraulic dredges are awesomely efficient movers of material as they maintain harbors and waterways for the Nation's waterborne commerce

Within this first project are five distinct tasks. The first is the most expensive and involves field research in open-water disposal areas. This research should answer the question of how to characterize a disposal site over a period of time. Is it really a barren wasteland, as some think? How fast do the flora and fauna recover after disposal? This task concentrates on studying individual, carefully selected sites currently being chosen from among several potential areas.

Once these pilot sites are selected, the research process starts with the examination of existing (baseline) conditions. Next the biological and water-quality changes occurring from carefully planned experimental disposal operations will be observed. Finally, everything that has been observed will be documented and the results of actually field-testing the hypotheses developed in the laboratory will be evaluated.

A second task within this project is to determine the fate of dredged material deposited in an open-water environment. How does the material disperse? Where does it settle out? Where does the suspended part (the turbidity-causing particles) end up? The answers to these questions are important because the specific movement pattern of the material often determines the effect on the aquatic environment.

The task will concentrate on the development of a predictive capability. At present the only viable approach toward determining sediment movement appears to lie with the use of mathematical models that simulate conditions during the time span between the instant of release and the moment the dredged material settles on the bottom. Long-term models will be developed to represent not only the transit time between discharge and settling, but also the redistributing effect of tides, storms, and circulation patterns.

Research will also be aimed at understanding and quantifying the effects of disposal on water quality and aquatic organisms. In terms of water quality, the immediate physical and chemical changes taking place in the water will be of primary concern. For the organisms living in that water the basic biological cause-effect relationships must be developed. Sediment geochemistry will be studied to determine the manner in which contaminants such as heavy metals in the sediments become available to cause adverse effects in the aquatic environment.

Heavy metals such as mercury, zinc and lead, along with such substances as pesticides, may have toxic effects on marine and freshwater life. Since the availability of such materials in various chemical forms in the marine environment is largely unknown, the investigations of their effects on water quality and aquatic organisms are beginning in the laboratory. Heavy metal contaminants may be tightly bound to the sediment particles physically or chemically or, at the other extreme, simply dissolved in the water mixed with the sediment. Preliminary indications based on laboratory results are that the only metals or nutrients released into the water from sediments passing through the water column are manganese and ammonia compounds. The field studies mentioned previously will be designed to verify these findings.

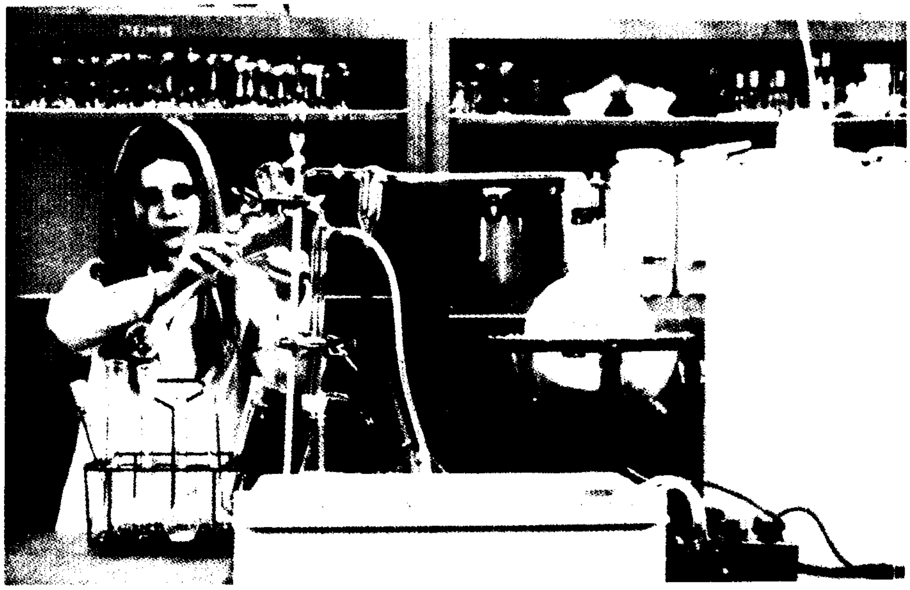

Fig. 3. Laboratory examination is being used to determine presence of pollutants in dredged material and possible effect on water quality and aquatic organisms

A separate task is attempting to define exactly what constitutes "polluted" sediment and its potential for causing problems within the environment when dredged. Sampling and analysis techniques will be developed to predict the probability of adverse effects and regulate operations accordingly.

Habitat Development

The second DMRP project contains three separate tasks designed to enhance the use by wildlife of lands associated with disposal of dredged material. While disposal operations in uplands and marshes have fundamental similarities to disposal in open water, different questions are posed.

Before the enhancement or creation of habitats for wildlife can be attempted, the many unknowns of the environmental impacts of land disposal must be determined. The first task in this project will address these problems. Upland and wetland environments are more diverse than those of open water, and since they are also man's habitat, there are socioeconomic factors to consider besides physical and biological ones. Aesthetics and land values must now enter into consideration, along with problems such as groundwater contamination.

Part of this task will be to pinpoint beneficial and adverse aspects so that as fully as possible monetary values may be given to them. Then a benefit-cost ratio on the monetary values can be used. For nonmonetary values, the factors involved must be isolated and evaluated qualitatively. The purpose is to put all factors into a usable form for decisionmaking.

The second task is concerned with creating artificial marshes and island habitats. Wetlands can be created by placing dredged material at an intratidal elevation in a shallow body of water and planting the appropriate marsh plant. The problem in creating new wetlands is to establish optimum growing conditions for marsh-type plants best able to tolerate the physical and chemical regime imposed by constant wetting and drying and any stresses which may be associated with the dredged material.

Fig. 4. Creation of marshes and wetland habitats is an extremely attractive, beneficial use for dredge material

That type of dredged material which is organic and nutrient-rich is well suited to help nature shape new wetlands, which are being destroyed faster than both man and nature can fashion them. Aside from the aesthetics of such areas, wetlands serve as wildlife sanctuaries, produce commercial and sport fishing, dampen storm effects, and function as natural traps for safe containment of materials which would otherwise become pollutants.

The third task within this project is aimed toward habitat development. This task involves manipulation of the environment for the benefit of a desirable species or community rather than actual creation of a habitat. Since one may attempt in one case to enhance a shallow underwater disposal site for marine grass growth while in another case to promote an upland game habitat or even a nesting and resting site for birds, this is a very site-specific task. The goal is to optimize a particular site for a specific type of lifestyle by controlling and changing such factors as elevation, size, or type of vegetation.

Fig. 5. Creating island sanctuaries for migratory waterfowl and endangered species need not conflict with protection of fisheries and water quality

In developing an island habitat, concentration is on the control of an area's fate either as unvegetated, open sand flats, or vegetated cover, depending on the habitat requirements of those species desired for the area. The species attracted will depend on how the pattern of colonization is regulated. The sequence of plant succession can be controlled to prolong a particular stage of growth for the benefit of the desired species.

Disposal Operations

This primarily engineering-oriented research project is subdivided into five tasks. The first, operation of confined land disposal sites, is producing immediate results for the Corps. Management of these sites must be optimized for two reasons. First, the facilities must be effective in preventing the unregulated spread of the often syruplike slurry (approximately 20 percent solids) into surrounding areas. Second, they must reduce the amount of suspended material in the effluent or runoff by trapping the solids and allowing only the water in the slurry to run over the weir or sluice.

Many problems are inherent in confining dredged material:

The integrity of dikes built on ground with poor foundations is difficult to maintain; yet the optimum containment sites are likely to exhibit the poorest soil qualities for this purpose.

Fig. 6. Diked disposal areas are being evaluated to increase their effectiveness

The dredged material stays fluid for extremely long periods unless treated or mechanically consolidated.

Stagnant water in the slurry sometimes generates odors and encourages breeding of mosquitos.

Containment areas that are too small or improperly designed permit the sediment to flow directly over the weir unless some physical means are present to slow down the flow.

The second task under this project includes devising techniques to densify deposited sediments. The great bulk of dredged material consists of silty clays, which present a problem in drainage and consolidation. Crusts up to 5 to 6 inches thick form over several feet of dredged material causing retention of its high water content. Densification means getting the water out. This can substantially increase the life expectancy of a disposal site by reducing the volume of the underlying solids. Several methods have been tested, including running a tracked vehicle back and forth to break up the crust and increase the rate of evaporation. Another promising possibility being explored is to plant certain reeds and canes in these sediments to reduce water content through transpiration.

The third task relates to the traditional treatment of waste materials as practiced by sanitary engineers in handling municipal and industrial sewage. The basic difference is that current treatment plants are geared to fairly constant flows. Dredged material, however, arrives on site sporadically and at an extraordinarily high loading rate. Therefore, current practices may or may not be amenable for treating dredged material contaminants. Conventional techniques are being assessed, and the most promising techniques will be pilot- and field-tested.

The fourth task is to predict the amount of turbidity caused by dredging and disposal operations and to seek methods of controlling it. Although the extent of the biological consequences of turbidity in open water are not fully understood -- and it is possible that the biological consequences of turbidity caused by dredged material disposal have been grossly overrated -- it cannot be denied that situations may occur where turbidity is biologically or aesthetically unacceptable, and some type of control is necessary. The goal of this task is to provide turbidity prediction and control techniques for those instances in which turbidity regulation is desirable or necessary.

The last task in this project, completing the picture, is to look at the dredging operation itself to see if modifying dredging practices can help alleviate disposal problems. Use of accessory equipment in the dredging or disposal operation is also being evaluated.

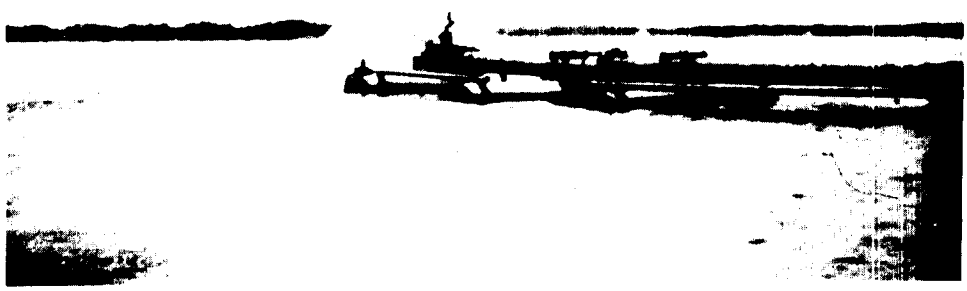

Fig. 7. Turbulence from dredging and disposal operations clouds adjacent waters with suspended particles of sediment that may or may not create adverse impacts on the environment

Productive Uses

This project consists of five tasks associated with potentially productive uses of disposal sites and/or dredged material, such as reclaiming strip-mined areas and providing waterfront land for recreational parks. These uses are in addition to the wildlife habitats on artificial islands mentioned earlier under another project.

In some instances the fine-grained sediment in slurry form might be transported inland for landfill at abandoned pits and quarries or for use in the enhancement of agricultural lands. In other cases the dredged material may be used to control coastal erosion. Many locations, including strip-mined areas, can be considered desirable disposal areas as well as be reclaimable land, since they are already in a degraded state.

Initial efforts will attempt to inventory and define the geographical extent of these potential disposal areas in relation to the locations where dredged material will normally be generated. Transportation to inland sites might be accomplished by pipeline or by hauling over railroads and highways. Unit trains of as many as 250 coal cars often return empty from shipping points to mining sites when they could possibly be filled with sediments and slurry suitable for landfill and reclamation use. This may sound like an oversimplified solution to a very complex technical and sociopolitical problem, but at least it illustrates the point.

The land improvement task is material-oriented rather than site-oriented and focuses on regions of the country where large sanitary landfills are required near the larger urban areas. The soil necessary to bury this landfill is not always available on site in these regions and has to be trucked in at considerable expense. Perhaps sediment, separated, drained, and stockpiled at disposal sites, could be transported to these landfills as substitutes.

The value in such a practice lies in the public attitude toward dredged material as a waste product of dredging and ever-increasing need for more valuable land for waste disposal. Since the dominant land use at sanitary landfills is disposal of wastes, the use of "dredged waste" to cover municipal waste should be publicly acceptable.

One of the associated problems to deal with is the quality of the "leachate" or liquid runoff from such areas induced by precipitation. The effects of the presence of dredged material in solid waste on ground water quality must be studied closely before this can be considered an acceptable disposal alternative.

As mentioned earlier, one of the most promising productive uses of dredged material is to provide landfill for water-oriented recreation areas such as islands, peninsulas, and shoreline parks. Although landfill composed of dredged material usually provides poor foundation conditions for many years, it will support uses such as ballfields, parking lots, parks, playgrounds, boat launching ramps, nature trails, overlooks, and conservancies.

Fig. 8. Shoreline parks and recreational areas on dredged material have been successful in some Corps Districts

Managing the Program

A program such as this requires a steering group to guide it, good coordination and lots of communication. Guidance is provided by a Program Planning Group consisting of the Chief of the Environmental Effects Laboratory; his special assistants for dredged material research, program development, and program management; the four DMRP project managers; the two DMRP coordinators; and a consultant from the WES Hydraulics Laboratory. This group is responsible for technical planning, coordination, and setting program priorities -- and it is backed up by Research Planning Groups functioning in each project.

The two full-time coordinators have been assigned to the DMRP for both internal coordination with Corps Districts and external coordination with other agencies. A separate group has been set up to increase the efficiency of communications through the use of technical reports and dissemination of current information on each research task via a widely circulated monthly newsletter.

The coordinators and publications make the research efforts known as quickly and widely as possible to the Corps Districts and other agencies. An even wider dissemination program is being developed so that the

Nation can more rapidly learn what dredged material consists of and its effects on the environment.

PARAMETER STUDY OF VARIABLES AFFECTING THE PERFORMANCE OF A HYDRAULIC PIPELINE DREDGE MODEL*

By

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INTRODUCTION

Any dredger knows the effects of line length, digging depth and grain size on solids output of a hydraulic dredge. In each case, as these parameters increase, production falls off. But what about the effect of various dredge pump designs (the "heart" of the dredge) on output?

It has been suggested to me by a number of knowledgeable dredge people that because of the large number of unknowns and variables involved with dredging, variations between different manufacturers' pump designs have very little effect on solids output. In other words, one pump is about as good as another (hydraulically) --- all things considered.

To test this hypothesis and the relative effects of other variables involved, a model of a hydraulic dredging system has been developed. This model is simply a set of equations which describe various elements of the system such as (1) an equation for the head-discharge relationship of the pump; (2) the pipe friction-head loss equations, etc. To facilitate the calculations an IBM 360/65 computer is employed. The advantage of the model is that one variable can be systematically varied while all others are held constant and the results quickly and economically compared.

MODEL DREDGE SYSTEM

The development of the model dredge, its limitations, and a computer listing are presented in detail elsewhere[1] and will not be repeated here. This discussion will concentrate on the results of a study of the major variables involved. In particular the objectives of this study were to:

(1) investigate the variability in solids output versus pumping distance (line length) for five different dredge pump designs. These tests to be conducted with a range of both dredge sizes and sediment sizes transported;

(2) investigate the variability in solids output versus digging

depth for the same five different pump designs; and

(3) consider the influence on dredge performance of other variables such as horsepower and suction pipe size.

Fig. 1 illustrates the large (27"ϕ), typical, hydraulic pipeline dredge used in this study. Other size dredge systems will be discussed later in this report. In all cases the pump and terminal output point are located at the water surface for convenience, although the model can accept any values for these elevations.

DREDGE PUMPS TESTED

Five different dredge pump designs (or test results) were employed to investigate this most important parameter's effects on solids output. Fig. 2 presents the characteristic head-capacity curves for these pumps (A, B, C, D, and E) in dimensionless form, where:

$$H_{dim} = \frac{gH}{\omega^2 D^2} \tag{1}$$

$$Q_{dim} = \frac{Q}{\omega D^3} \tag{2}$$

H = total dynamic pump head, L

ω = pump rotative speed, (rad/sec), L/T

D = characteristic pump size, (impeller diameter), L

Q = pump volumetric flowrate, L^3/T

g = gravity constant, L/T^2

The efficiency and dimensionless brake horsepower curves are shown in Fig. 3 and 4 for these same pumps, where:

$$E = \text{pump efficiency} = \frac{Q\gamma H}{550\ \text{BHP}} \tag{3}$$

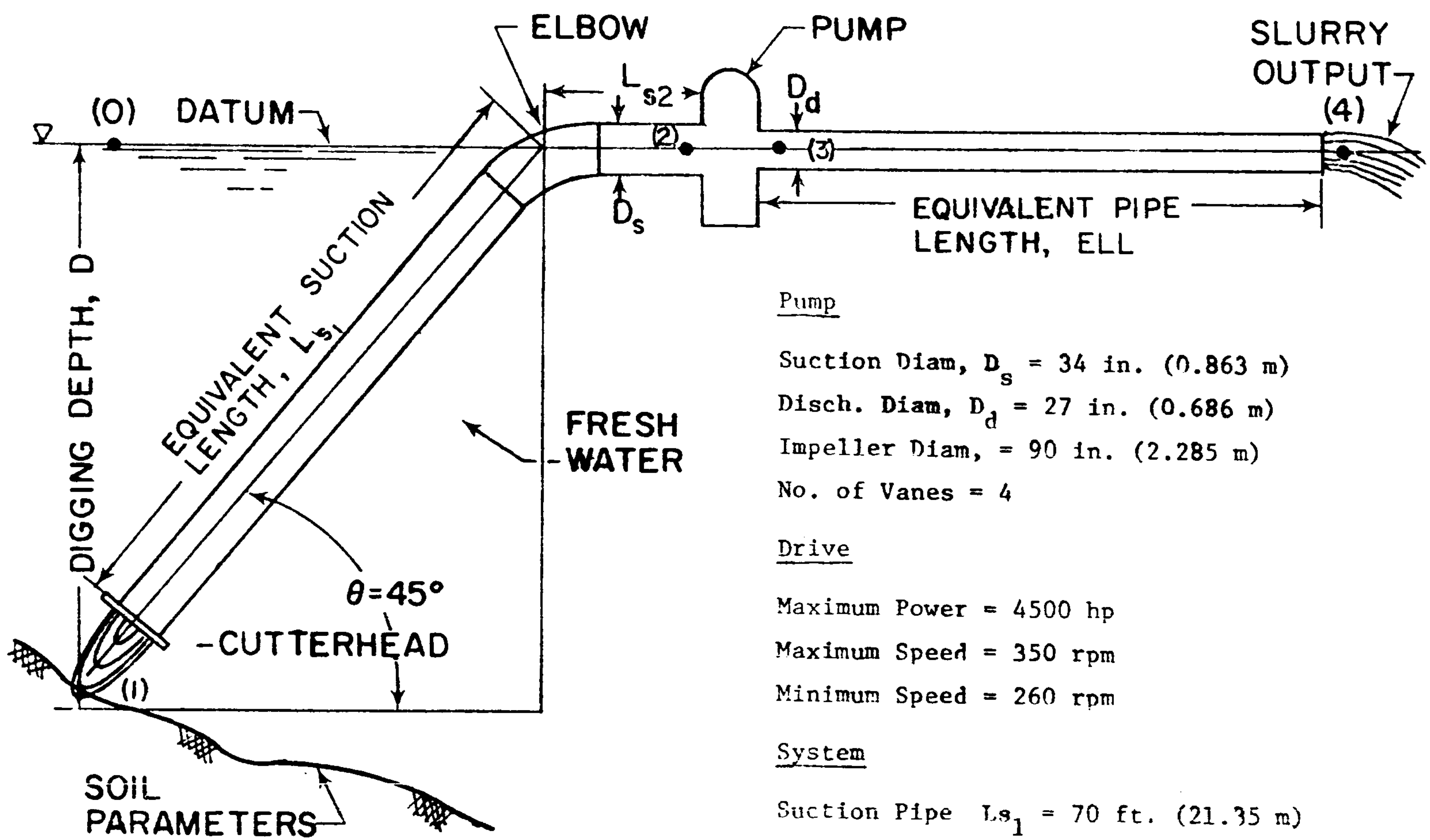
$$BHP_{dim} = \frac{gQH}{\omega^3 D^5} \tag{4}$$

γ = unit weight of fluid transported, F/L^3

In Fig. 5, the required cavitation limits are shown for each test pump, where:

$$\sigma_c = \text{cavitation index} = \frac{\text{NPSHreq'd}}{H}$$

NPSHreq'd = the required net positive suction head above vapor pressure to prevent cavitation



Pump

Suction Diam, D_s = 34 in. (0.863 m)

Disch. Diam, D_d = 27 in. (0.686 m)

Impeller Diam, = 90 in. (2.285 m)

No. of Vanes = 4

Drive

Maximum Power = 4500 hp

Maximum Speed = 350 rpm

Minimum Speed = 260 rpm

System

Suction Pipe Ls_1 = 70 ft. (21.35 m)

Ls_2 = 10 ft. (3.05 m)

Discharge Pipe Leq = variable

Soil

Mean Diameter, d_{50} = 0.42 mm

Fig. 1 ILLUSTRATIVE EXAMPLE OF HYDRAULIC DREDGE SYSTEM

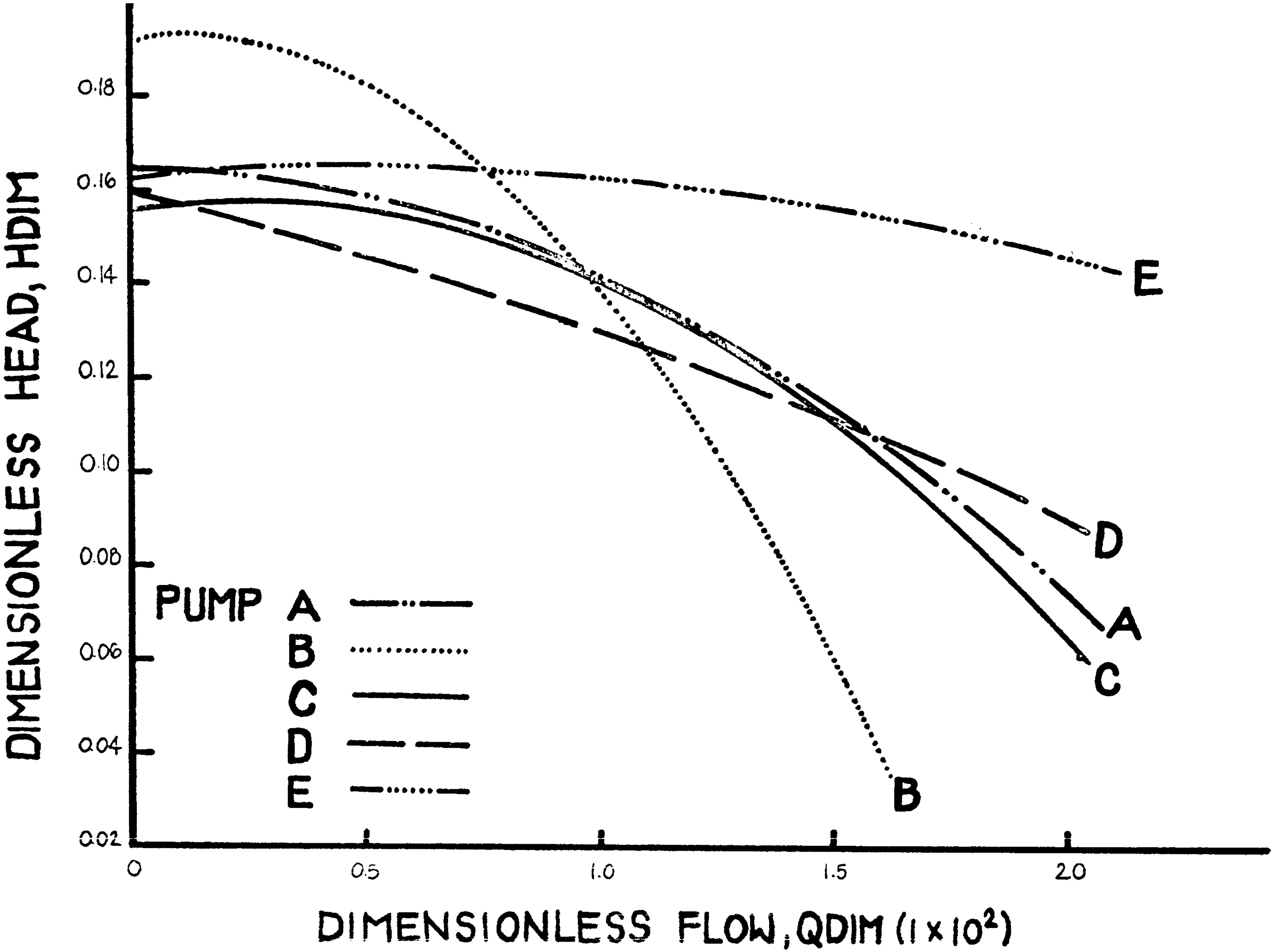


Fig. 2 DIMENSIONLESS HEAD CHARACTERISTIC CURVES FOR PUMPS CONSIDERED

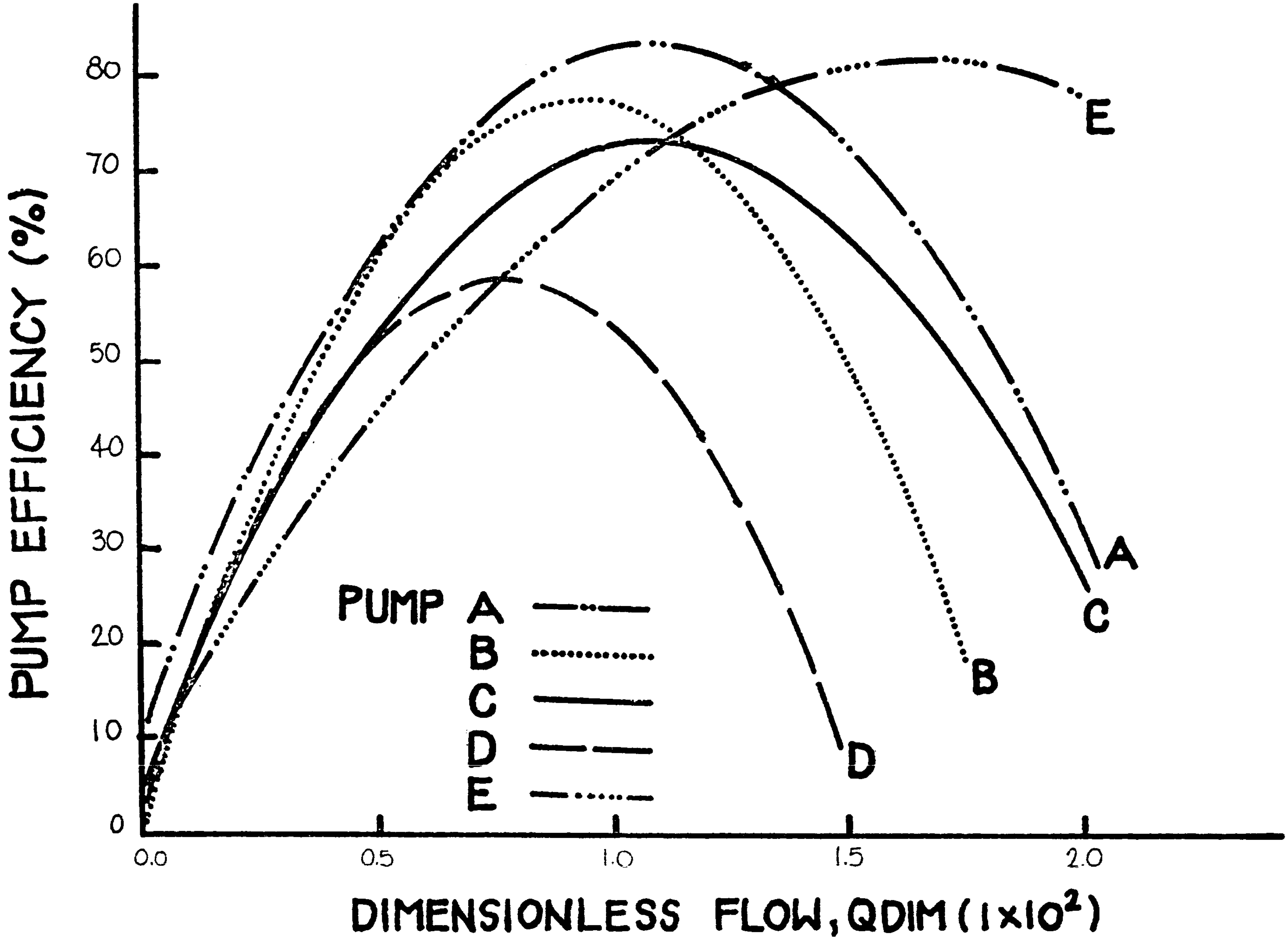


Fig. 3 EFFICIENCY CURVES FOR PUMPS CONSIDERED

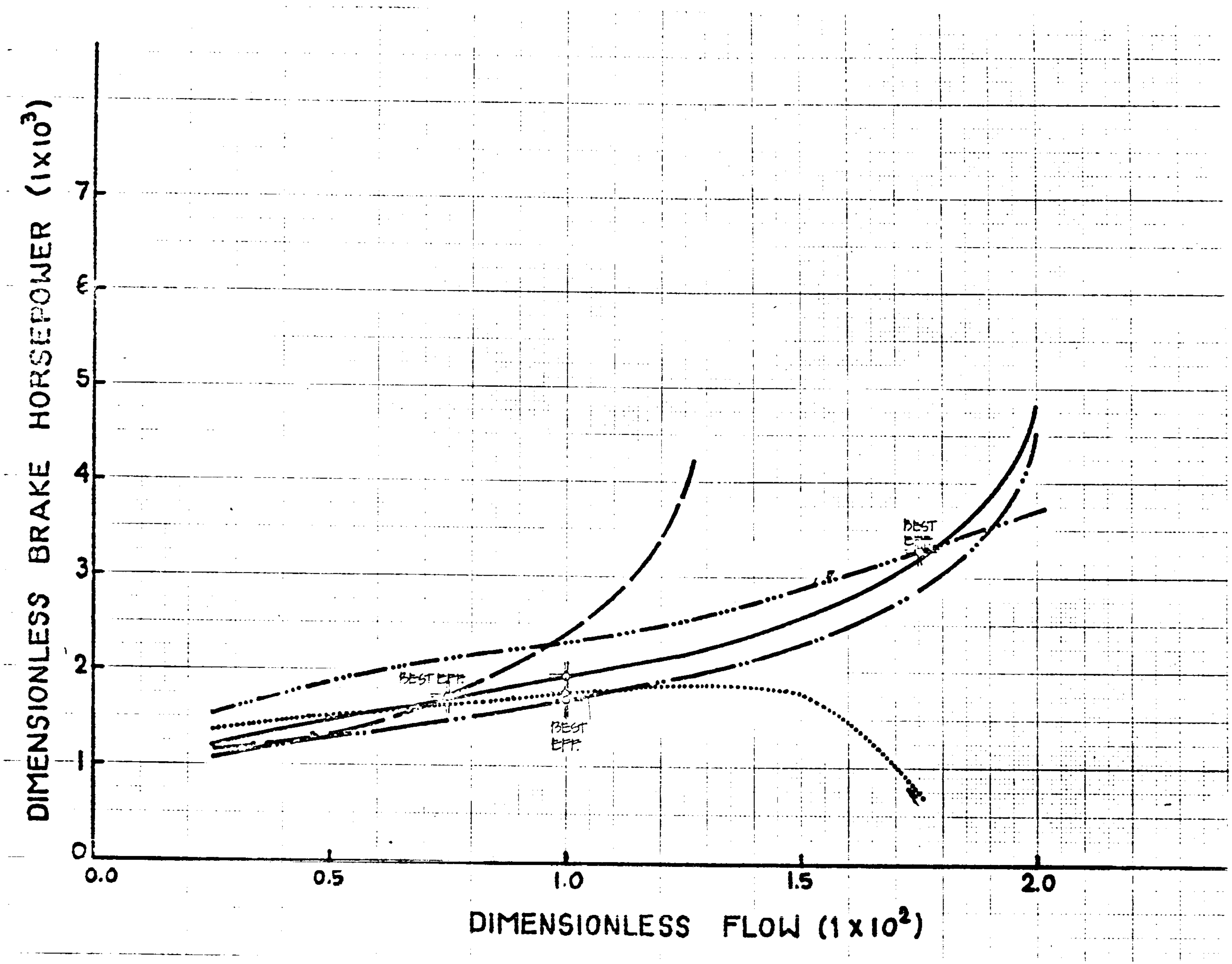


Fig. 4 DIMENSIONLESS BRAKE HORSEPOWER CURVES FOR PUMPS CONSIDERED

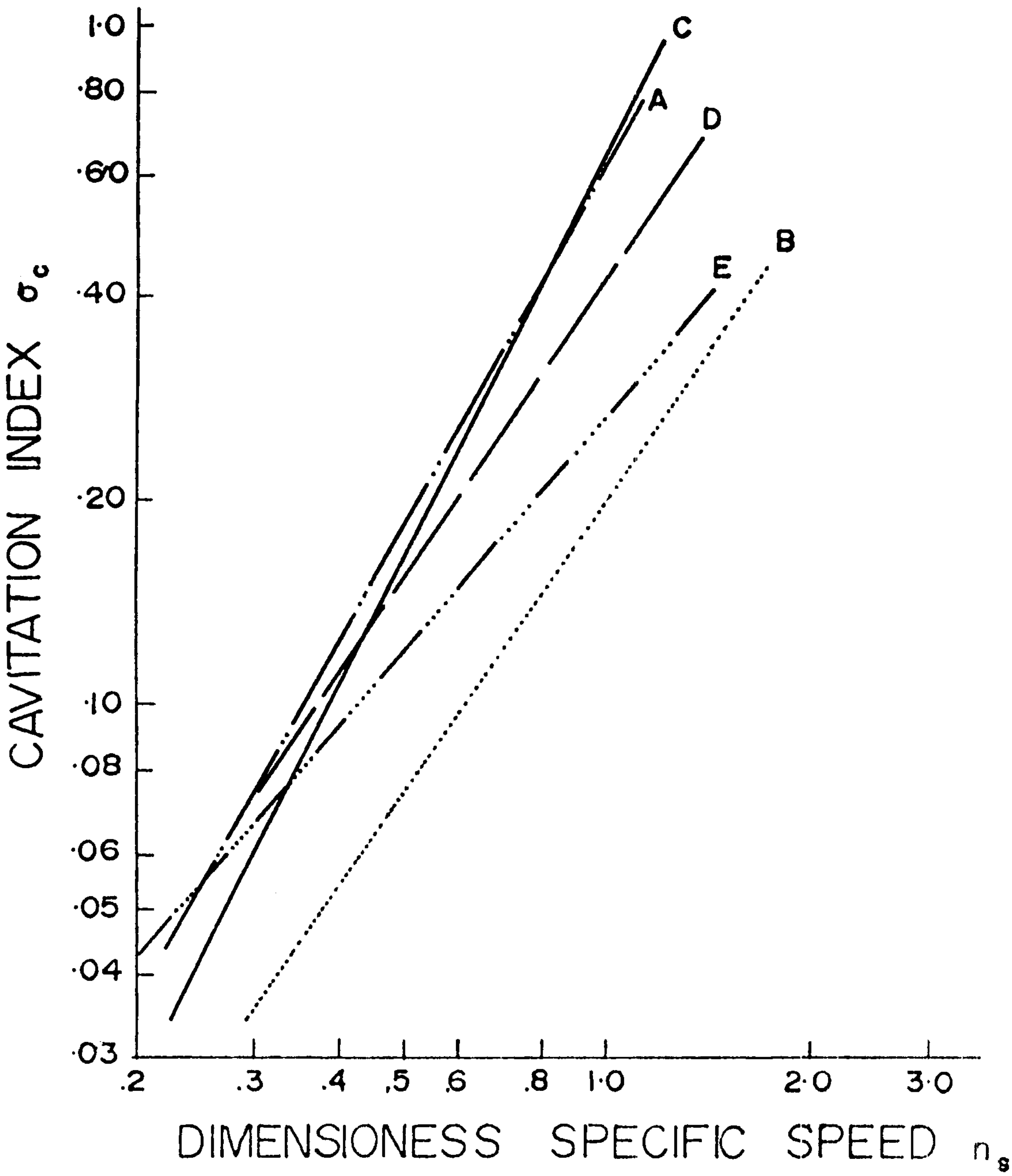


Fig. 5 DIMENSIONLESS CAVITATION LIMITATION CURVES FOR PUMPS CONSIDERED

$$n_s = \text{dimensionless specific speed}$$

$$= \frac{\omega Q^{1/2}}{(gH)^{3/4}} \qquad (6)$$

Because of their dimensionless nature, Figs. 2, 3, 4, and 5 can be employed (for approximation purposes) with any homologous-sized pump running at any speed.

Computer, curve-fit programs have been employed to determine the basic equations for these curves so that they may be readily employed in computer computations. Fig. 6 presents all the coefficients employed in these equations for each pump along with some remarks concerning the origin of the data. The head and efficiency curves were second order polynomials while the cavitation index varied to some power of the specific speed.

DREDGE SYSTEM LIMITATIONS

The solids output of hydraulic pipeline dredges is basically limited by the following four factors which are the combined result of many inter-related and independent variables:

(1) Horsepower limitation (maximum line length)
(2) Cavitation limitation (maximum digging depth)
(3) Concentration limitation (prevent line plugging)
(4) Dislodgement limitation (maximum solids output)

The analytical model considered herein is limited to the first two factors above since technical information is not available to permit scientific estimates of line plugging or dislodgement limitations for various suction systems and dredge operation procedures. A detailed discussion of these limiting factors has recently been approved for publication.[2] Computation procedures for the horsepower and cavitation limitations would be as described briefly below.

Horsepower (Line Length) Limitations

The pump head characteristic curve(s) would be available from a certified performance test. Application of the basic work-energy principles from point (0) to point (4) in Fig. 1 would result in the entire pump head being required to overcome pipe friction and turbulence head losses plus provide one discharge velocity head at the pipe exit. The pump head curves would need to be corrected for concentration and size of material transported.[3]

PUMP	HORSEPOWER LIMITATIONS							CAVITATION LIMITATIONS		
	HEAD COEFFICIENTS			EFFICIENCY COEFFICIENT			REMARKS	CAVITATION COEF.		REMARKS
	HA1	HA2	HA3	EA1	EA2	EA3		NA1	NA2	
A	0.164	0.078	-227.8	0.1025	834.5	-6140		0.635	1.780	EXTRAPOLATED FROM ACTUAL TEST ON 6"φ DREDGE PUMP @ 1182 RPM, BY GRAPHICAL METHOD
B	0.192	1.686	-695.6	0.0002	165.4	-8763	FROM MANUFACTURE PUMP CURVES FOR 16"X16"X46" PUMP @ 500 RPM	0.205	1.435	ESTIMATED FOR W.S. VENTURE COMPUTATIONS, NO ACTUAL TEST DATA AVAILABLE
C	0.162	0.684	-239.4	0.1042	105.9	-4421	COMPOSITE OF MANY PUMPS - EFFICIENCY RAISED FOR LARGER DESIGNS - ORIGINAL COMPUTATION.	0.657	1.966	COMPOSITE OF MANY PUMPS - OBTAINED BY LEAST SQUARE - ORIGINAL COMPUTATION
D	0.139	-2.421	-50.3	0.0415	143.2	-9321	FROM CDS TEST ON 8"X6" PUMP @ 700 RPM	0.430	1.475	EXTRAPOLATED FROM ACTUAL TEST @ 700 RPM (26") BY GRAPHICAL METHOD
E	0.163	0.773	-80.3	0.0548	92.0	-2757	FROM MANUFACTURER'S CURVE	0.270	1.150	EXTRAPOLATED FROM ACTUAL TESTS OF 6"φ PUMP "A" @ 1752 RPM BY GRAPHICAL METHOD

$$HDIM = HA1 + (HA2)\,QDIM + (HA3)\,QDIM^2$$

$$EFF. = EA1 + (EA2)\,QDIM + (HA3)\,QDIM^2$$

$$\sigma_c = NA1\,(n_s)^{NA2}$$

Fig. 6 TABLE OF COEFFICIENTS FOR DIMENSIONLESS PUMP CHARACTERISTIC AND CAVITATION EQUATIONS

Relationships for slurry head loss in pipes would also be required which depend upon such factors as pipe roughness, length, flow rate (velocity), solids concentration by volume, grain size and distribution, and others[4]. For the maximum brake horsepower available, the above relationships can be combined by a trial-and-error process to determine total solids output rate (volume per unit time) for various line-lengths (pumping distances) and volume concentrations of solids transported. The envelope of these concentration curves gives the optimum (maximum) solids output -- an example of which is shown in Fig. 7 for a given dredge size and soil size. The maximum brake horsepower available is related to the pump-characteristic curves (Figs. 2, 3, and 4), hence one pump design may give completely different results for solids output versus pumping distance. Thus, Fig. 7 will form one basic system characteristic for use in comparing different dredge pump designs under constant available horsepower conditions. It should be mentioned here that solids output rates are for actual output and not in situ volumes of materials dredged.

Cavitation (Digging Depth) Limitation

Similarly, pump NPSH required curves would be available from actual tests. Application of the work-energy principle from point (0) to point (2) in absolute energy terms would permit determination of the NPSH available in the suction system to suppress cavitation. As the digging depth, slurry concentration and flowrate all increase, the NPSH available decreases. Slurry effects on NPSH required and suction pipe head losses must also be taken into account (see previous referenced articles). Equating available and required NPSH values permits determination of the maximum digging depth for a given volume concentration of solids present and solids output rate. Fig. 8 illustrates the results for the same sample dredge system shown in Fig. 1 and Fig. 7.

The maximum digging depth is related to the pump cavitation curves (Fig. 5), hence one pump design may give completely different solids output versus digging depth results. Fig. 8 will provide the other basic form for comparing different dredge-pump designs.

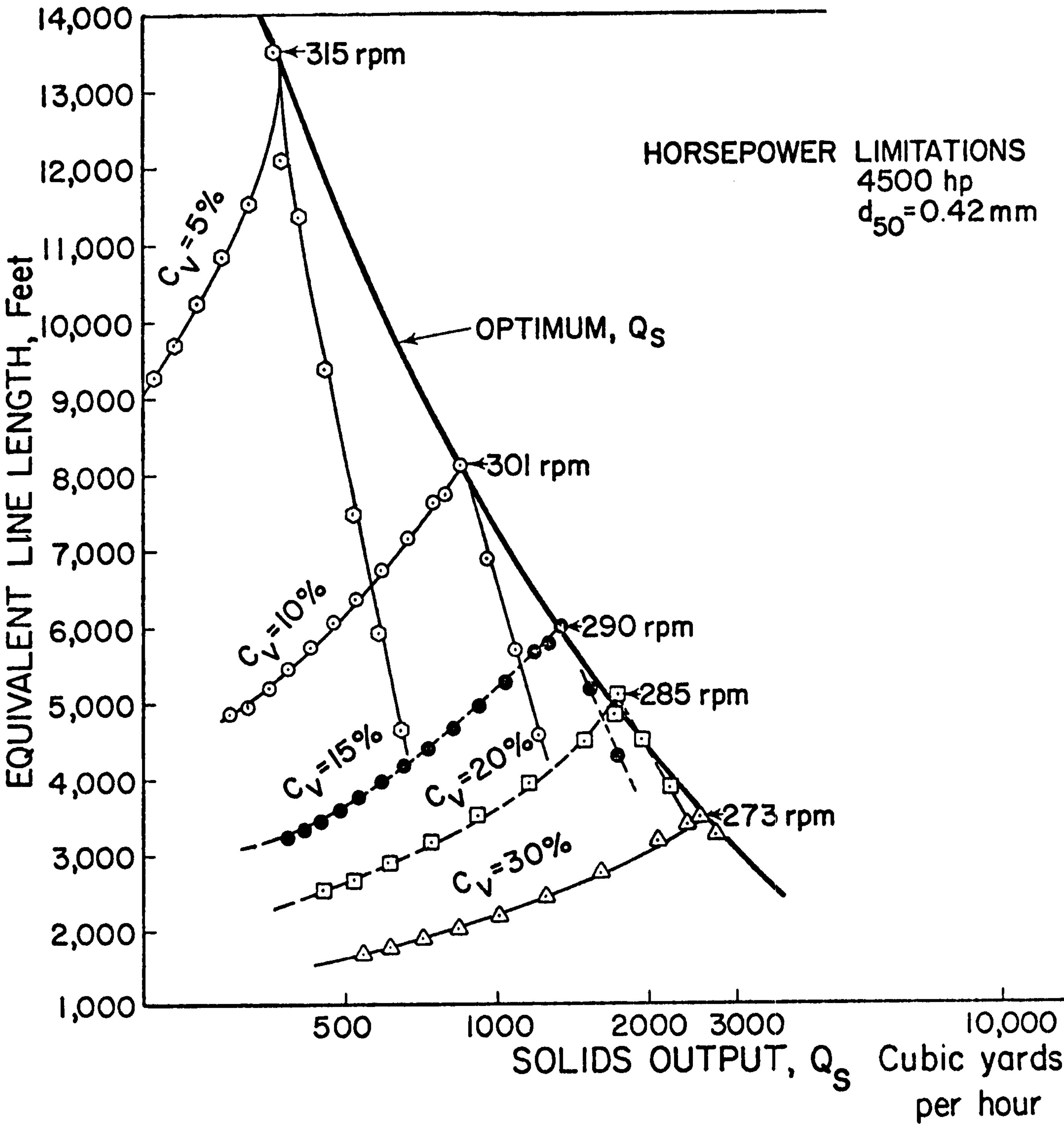


Fig. 7 DREDGE OUTPUT LIMITED BY DRIVE HORSEPOWER AVAILABLE

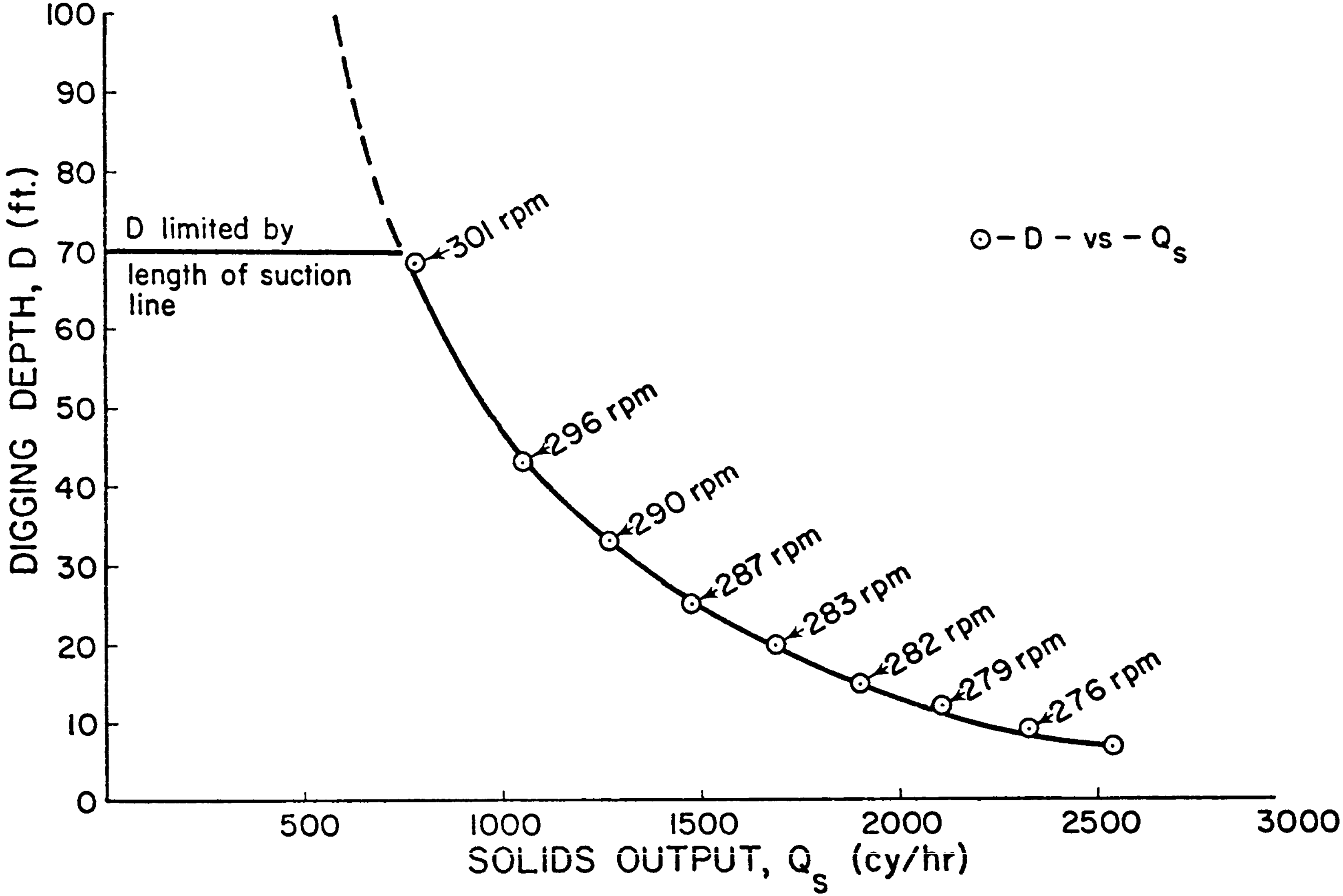


Fig. 8 CAVITATION LIMITATION (MAXIMUM SOLIDS OUTPUT -VS- DIGGING DEPTH)

TEST RESULTS

Test Program Design

Of primary concern was the influence of the various pump designs of Fig. 6 on solids output for a given pumping distance and digging depth. Consequently, it was felt to be important to also study the relationship between the size of the dredge and these variables. Three typical dredge sizes were employed:

Large - 27-inch discharge pipe

Medium - 16-inch discharge pipe

Small - 6-inch discharge pipe

The key dimensions for these dredges are shown in Fig. 9. In addition, the results were determined for four different sand sizes (d_{50} = 0.10, 0.23, 0.42, and 2.0 mm). Other variables such as available maximum horsepower and suction pipe size were also considered for the large dredge system.

Horsepower Limitation -- Effects of Pump Design

For the large, 27-inch dredge system with d_{50}=0.42 mm sand, the horsepower limitation produced the results shown in Fig. 10 for the five test pumps considered. It appears that dredge pump A produces the largest solids output over the entire range of pumping distances considered and is significantly different in results than the lower curves for pumps B or E. From 5000 to 7500 feet, pump A produces over 1000 cubic yards per hour more than pump B. Or, looking at it from another way, for outputs from 1000 to almost 2500 cy/hr, pump A delivers these quantities over 2000 feet further in distance.

Similar results were obtained for the small dredge as demonstrated in Fig. 11, although the order of ranking of pumps in terms of best production was different. Detailed results for all dredge systems and sediment-size combinations are not presented herein but can be found in the final report.[5] A summary of these results is tabulated and presented as Fig. 12. In all cases, significant differences in outputs were evident between the "best" and "worst" pumps; however, these pumps varied with dredge-system size and sediment size. For example, although pump B was "near best" for the medium and small dredges, it proved to be the "worst"

ITEM	VARIABLES		
Dredge Size	Large (27"Φ)	Medium (16"Φ)	Small (6"Φ)
Impeller	90"	42"	24"
Suction	34"	18"	8"
Max RPM	400	500	1000
Max HP	4500	500	150
Horsepower available	3000, 4000, 4500 5000, 6000, 7000		
Suction pipe size	27", 30" 32", 34" 36", 38"		
Sediment Size	SAND 0.10, 0.23, 0.42, 2.0 millimeters		
Pump Designs	A, B, C, D, E		

Horsepower Limitation (ELL-vs-Q_s) } * Pump designs, sediment size
Cavitation Limitation (DD-vs-Q_s) } suction pipe size, horsepower range

Fig. 9 DESIGN OF MODEL TEST PROGRAM

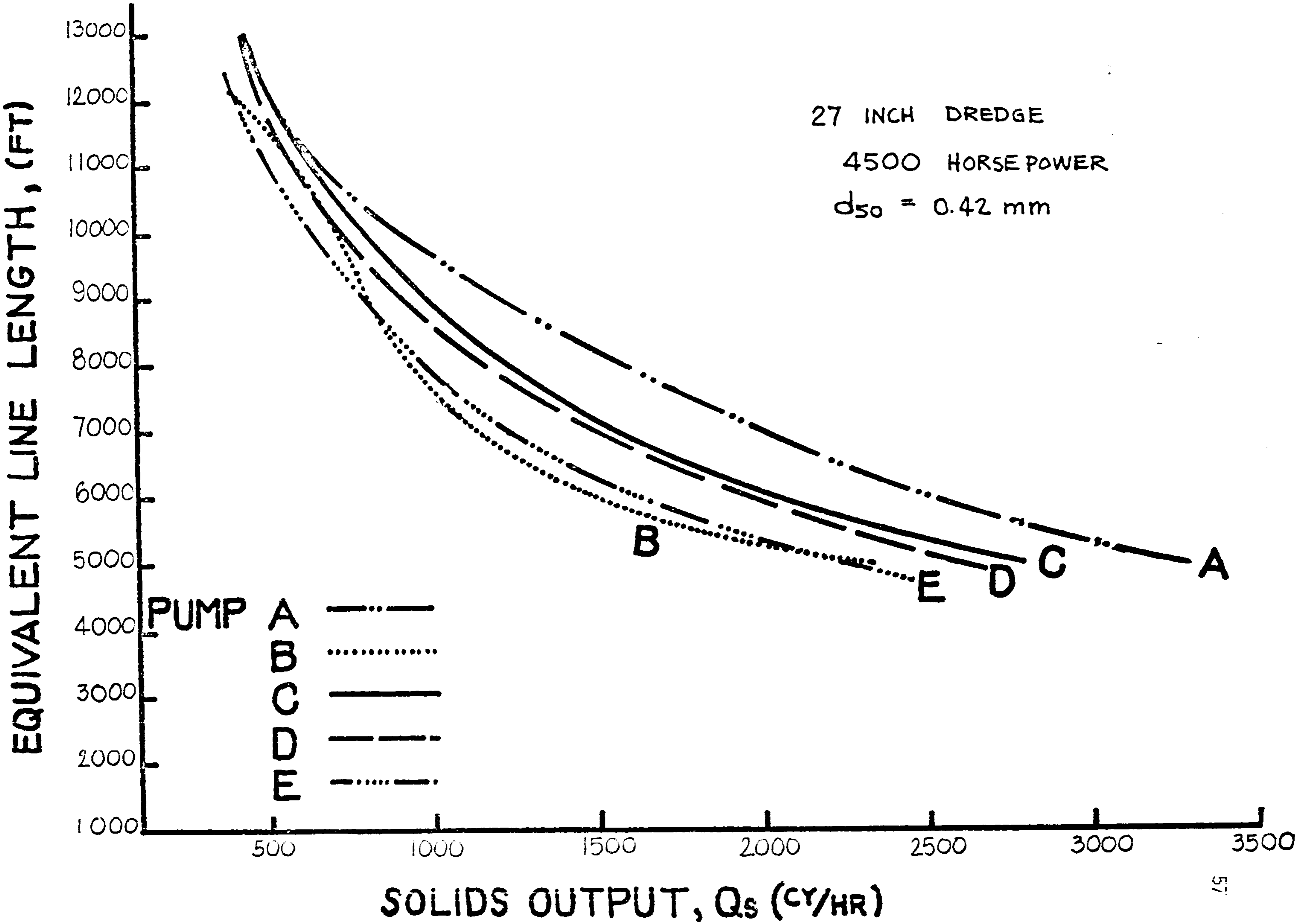


Fig. 10 HORSEPOWER LIMITATION, PUMPING DISTANCE VERSUS SOLIDS OUTPUT FOR PUMPS CONSIDERED IN LARGE, 27" DREDGE SYSTEM

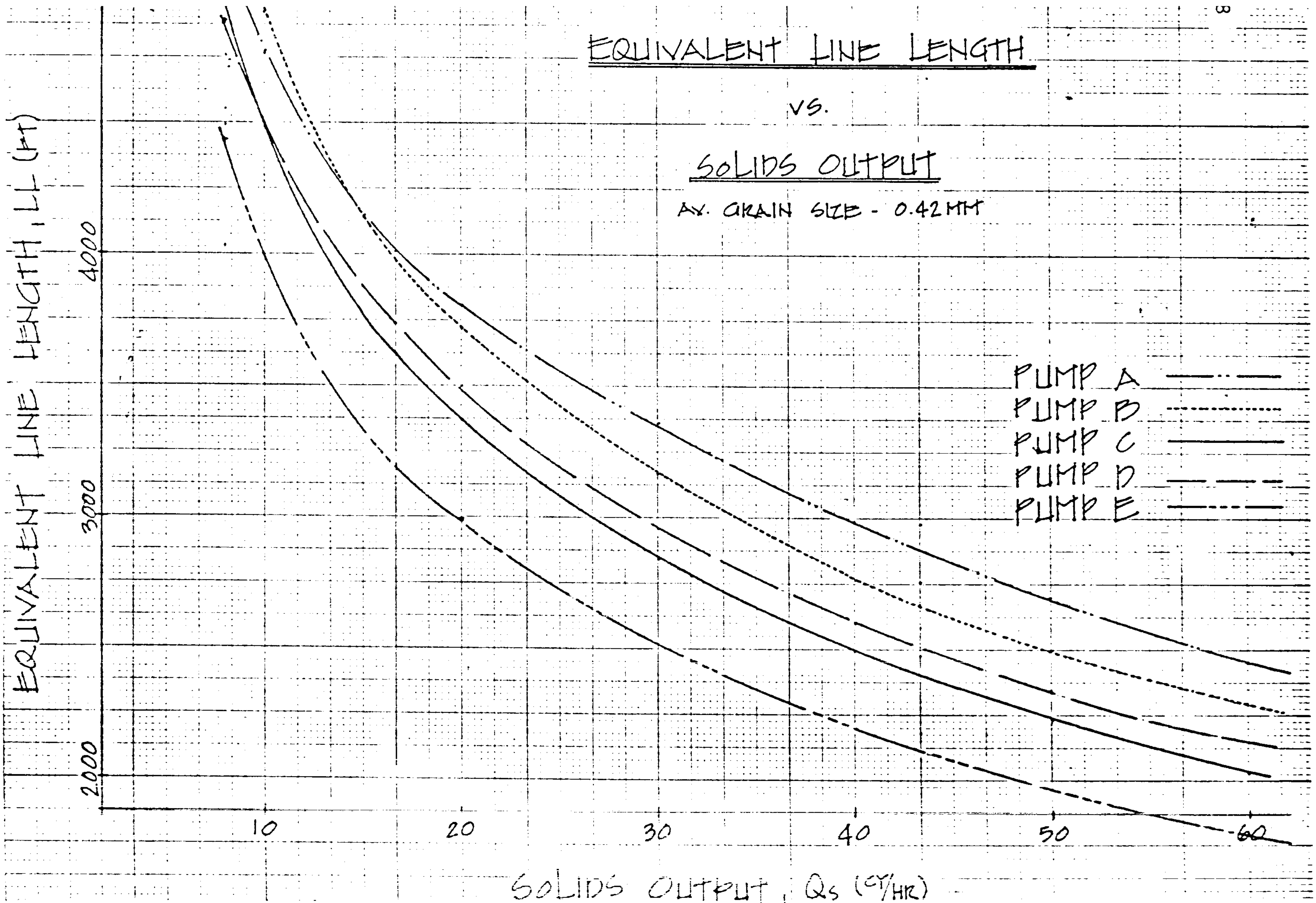


Fig. 11 HORSEPOWER LIMITATION FOR PUMPS CONSIDERED IN SMALL, 6" DREDGE SYSTEM

SUMMARY OF RESULTS

PUMP SIZE		AVERAGE GRAIN SIZE 0.1 MM	0.23 MM	0.42 MM	2.0 MM
BEST	SMALL	A & B	A & B	A & B	A & B
	MEDIUM	B	B	A & B	A & B
	LARGE	A	A	A	A
WORST	SMALL	E	E	E	E
	MEDIUM	D & E	D	D	D
	LARGE	B	B & E	B	B

	IMP. DIA.	DIS. DIA.	SUC. DIA.
SMALL	24"	6"	8"
MEDIUM	42"	16"	18"
LARGE	90"	27"	34"

Fig. 12 SUMMARY OF RESULTS FOR HORSEPOWER LIMITATION ON TEST PUMPS CONSIDERED

when used in the large dredge for all sediment sizes considered.

Pump A proved close to superior for all conditions tested. However, as shown in Fig. 2, pump A was average and pump E developed the largest head for most of the flowrate range considered. Pump E proved to be the "worst" or close to it in most instances even though its efficiency curve reached 80% at the peak (Fig. 3). Thus a paradox has developed between what appears to be excellent pump performance (pump E in Figs. 2 and 3) and the relatively poorer solids output (pump E in Figs. 10, 11, and 12) that resulted when this pump was used in the dredge.

The key to this dilemma is how the dredge pump curves interact with the dredging system curves or, i.e., <u>where</u> the pumps operate on their characteristic curves. In order to understand this relationship, Figs. 2 and 3 have been redrawn showing the range of pump operation on the head-capacity curve (Fig. 13) and efficiency-capacity curve (Fig. 14) for the horsepower limitation on system performance as shown in Fig. 10. It now becomes apparent that pump E is forced to operate well back on its head curve (Fig. 13) in order to stay within the horsepower limitation, and consequently the pumping efficiencies are quite low (Fig. 14), and the lower flowrates produce low solids output. In contrast, pump A operates at or near the best efficiency point, which, since it is higher than the other pumps, allows pump A to deliver more solids output per unit time. Consequently, knowledge of <u>both</u> (1) pump characteristics and (2) system operation are essential to determine the optimum dredge pump for a given dredging system.

The effects of two other variables on the horsepower limitation have also been included for consideration in this report. The effects of varying the maximum horsepower are shown in Fig. 15 for the large dredge and pump A. As expected, more solids production results as maximum horsepower increases. Also shown are the speeds that pump A requires to achieve these results. Perhaps, if more horsepower were available, pump E dredge production may significantly improve as system performance moves out toward the best efficiency point (Fig. 14). Fig. 16 shows the expected results for the effects of representative grain-size tests on solids output. The need and location of booster pumps can readily be determined from such information.

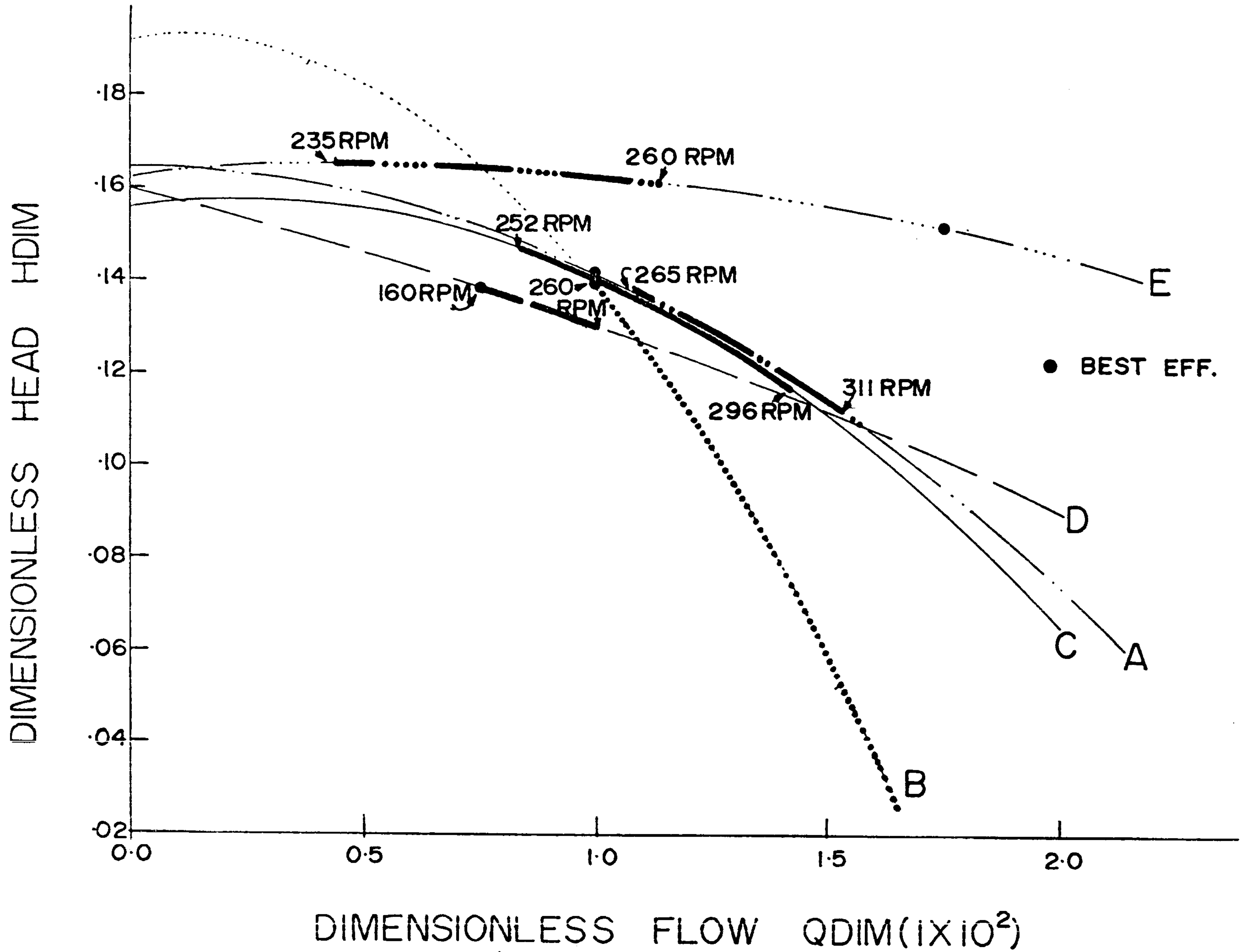


Fig. 13 RANGE OF PUMP HEAD OPERATION FOR SYSTEM PERFORMANCE SHOWN IN FIG. 10

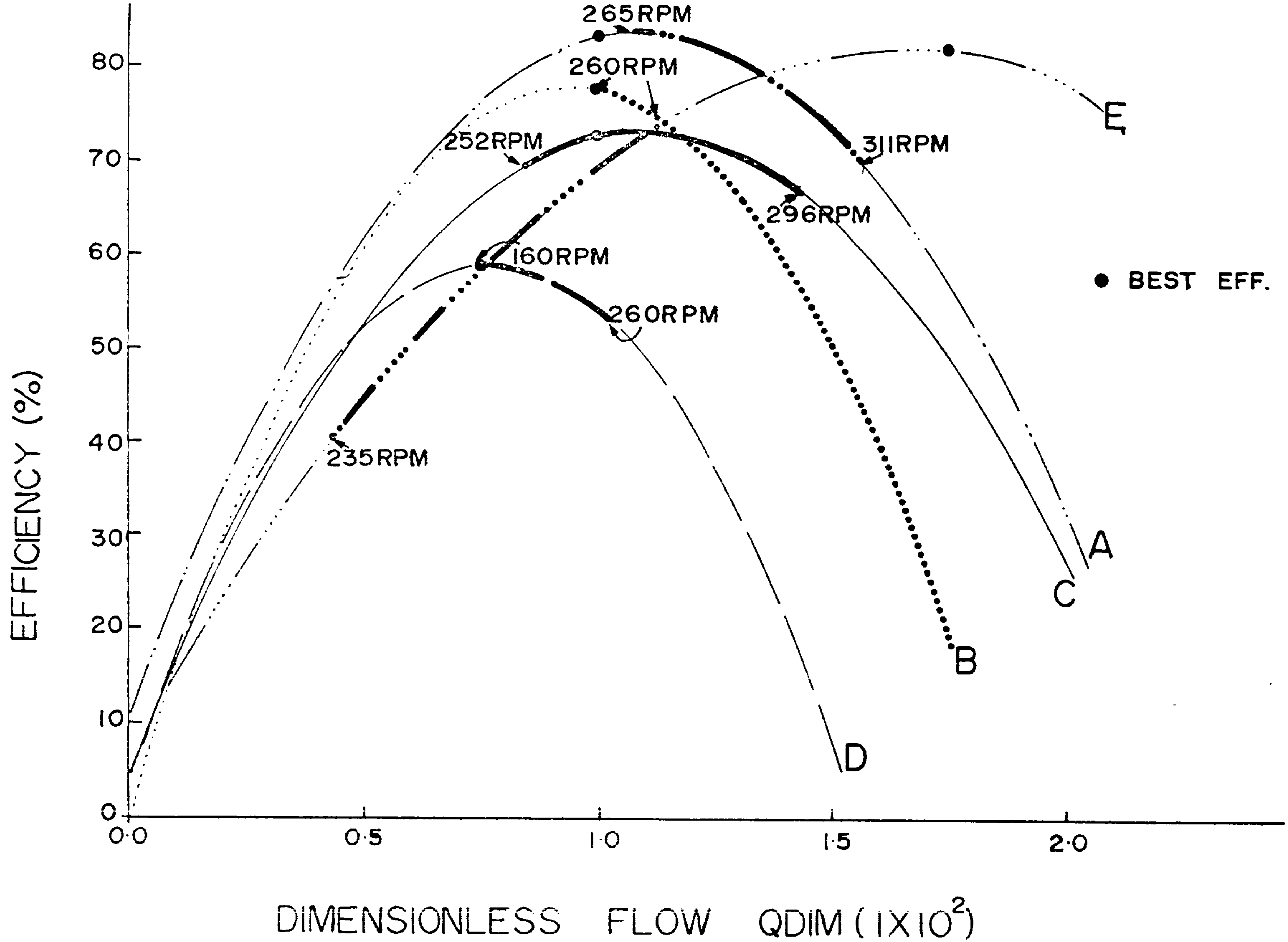


Fig. 14 RANGE OF PUMP EFFICIENCY OPERATION FOR SYSTEM PERFORMANCE SHOWN IN FIG. 10

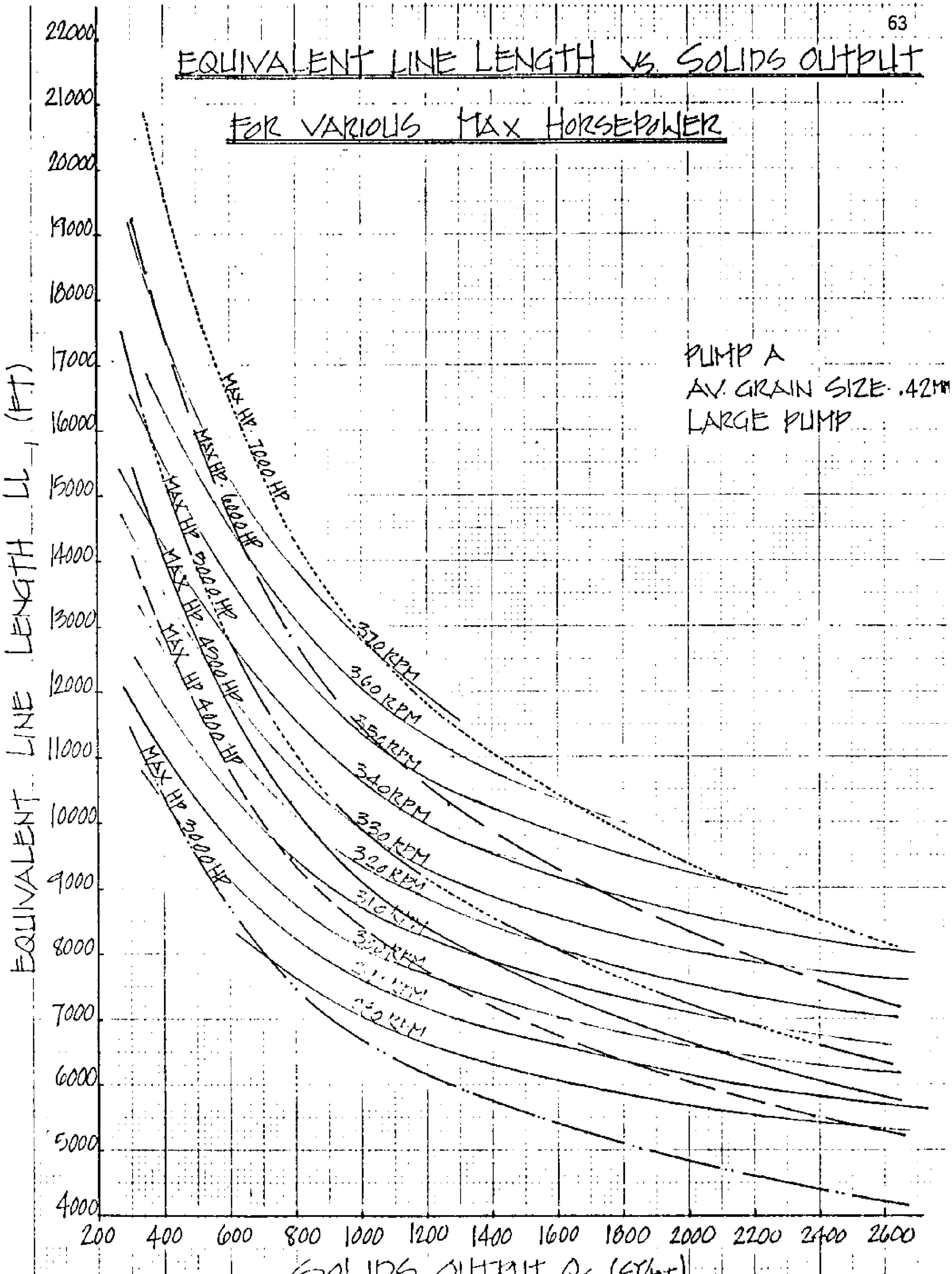


Fig. 15 EFFECTS OF VARIOUS HORSEPOWER LEVELS ON THE LARGE DREDGE HORSEPOWER LIMITATION

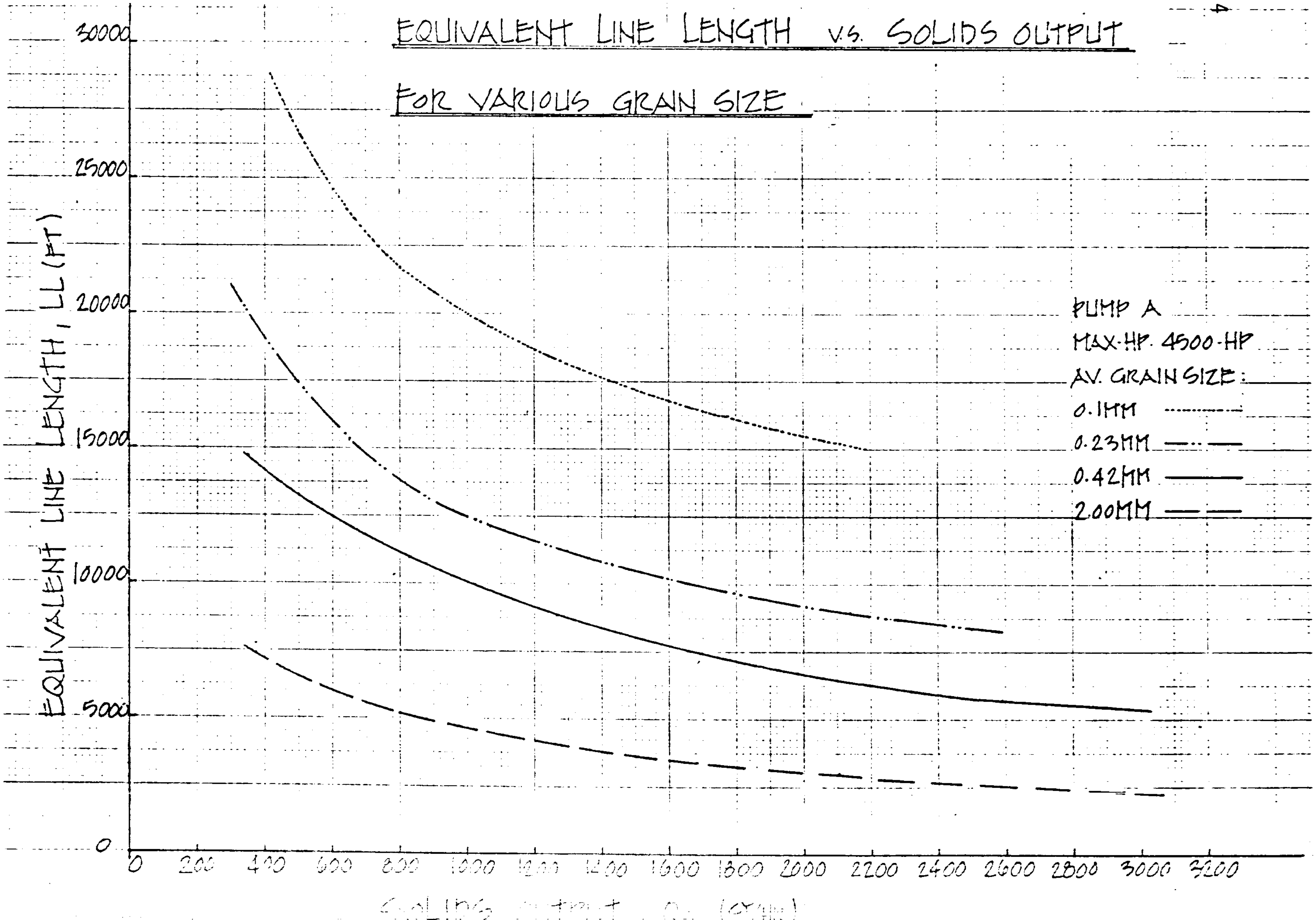


Fig 18 EFFECTS OF SOIL MEDIAN GRAIN SIZE ON THE LARGE DREDGE HORSEPOWER LIMITATION

Cavitation Limitation -- Effects of Pump Design

For the 27-inch dredge with 0.42 mm sand, the cavitation (digging depth) limitation produced the results shown in Fig. 17 for the five test pumps. Dredge pump B produced the highest solids output over the entire range of digging depths considered and is considerably different in results from pumps A and D, which produced the lower curves. Pump B created about twice as much solids output per unit time than pumps A or D, over the entire digging depth range.

Similar results were obtained for the other dredges considered, although the order of ranking of pumps in terms of best production was different. In all cases, significant differences in outputs were evident, primarily due to differences in pump design to produce lower NPSH requirements. Detailed results for all dredge systems and sediment-size combinations are not presented herein but can also be found in the final report.[5]

Many other variables influence the cavitation limitation on digging depth. Two examples are included in this report for consideration. In Fig. 18, the effects of various sediment sizes are considered on solids output for various digging depths (pump C). As expected, production generally decreased as grain size increased. However, production with 0.23 mm sand appeared slightly higher than with 0.1 mm sand. No explanation can be found for this result at this time, except possibly some discrepancy in the empirical head-loss equations employed to compute slurry friction losses in the suction pipe.

Dredgers have also employed increases in the suction pipe size to reduce losses and increase output. Fig. 19 shows the model test results for the large dredge system with 27-, 30-, 32-, 34-, 36-, and 38-inch diameter suction pipes (pump C). The 34-inch diameter produced the largest solids output over the complete digging depth range. Smaller sizes generated higher velocities to keep sediment in suspension, but created higher head losses and reduced the available NPSH to repress cavitation. On the other hand, larger pipe sizes generated lower head losses but required smaller sediment concentrations to keep sediment in suspension for transport. Hence, the 34-inch diameter is an optimum value. Interestingly, the actual dredge from which the large dredge

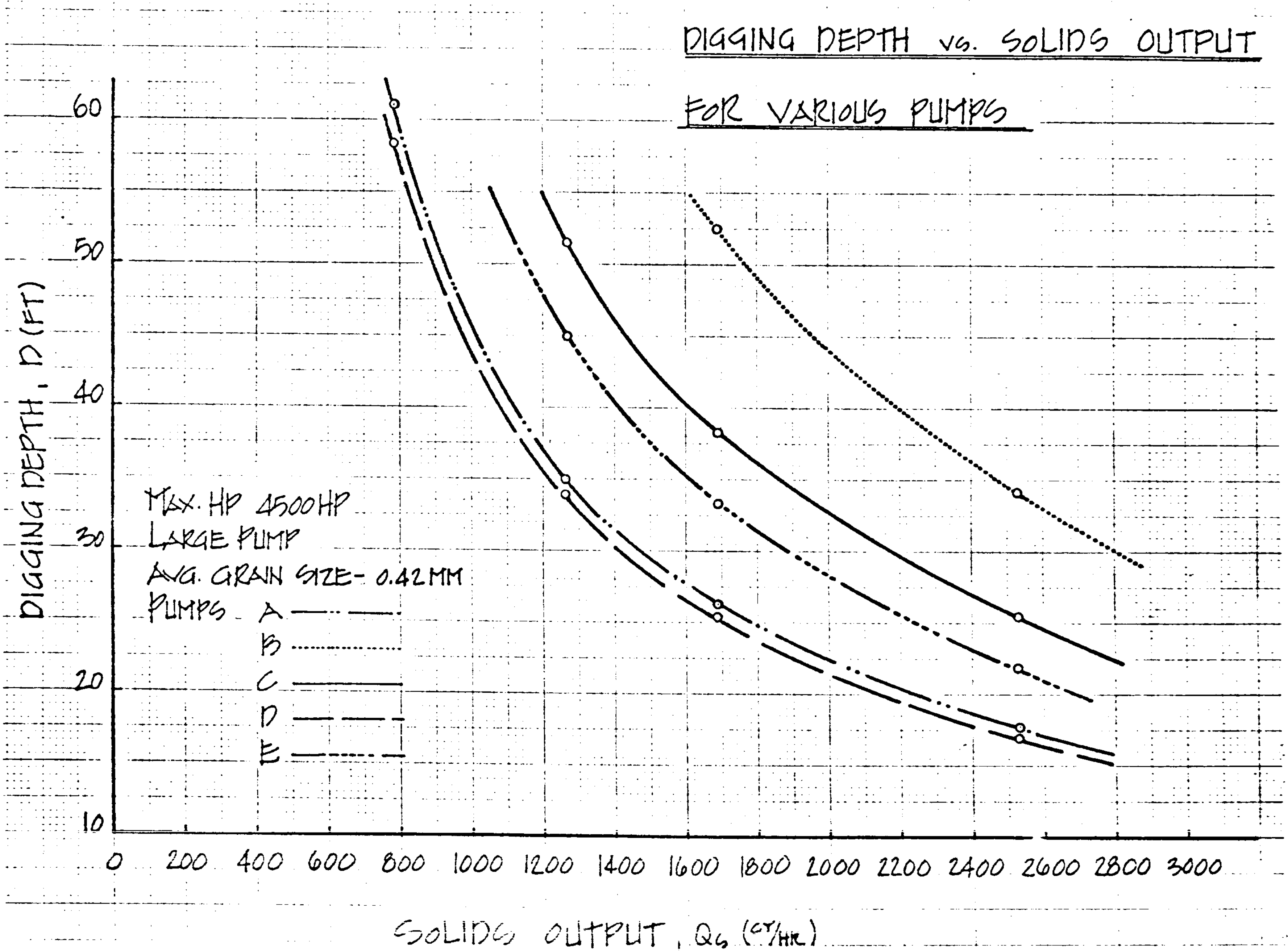


Fig. 17 CAVITATION LIMITATION, DIGGING DEPTH VERSUS SOLIDS OUTPUT FOR PUMPS CONSIDERED

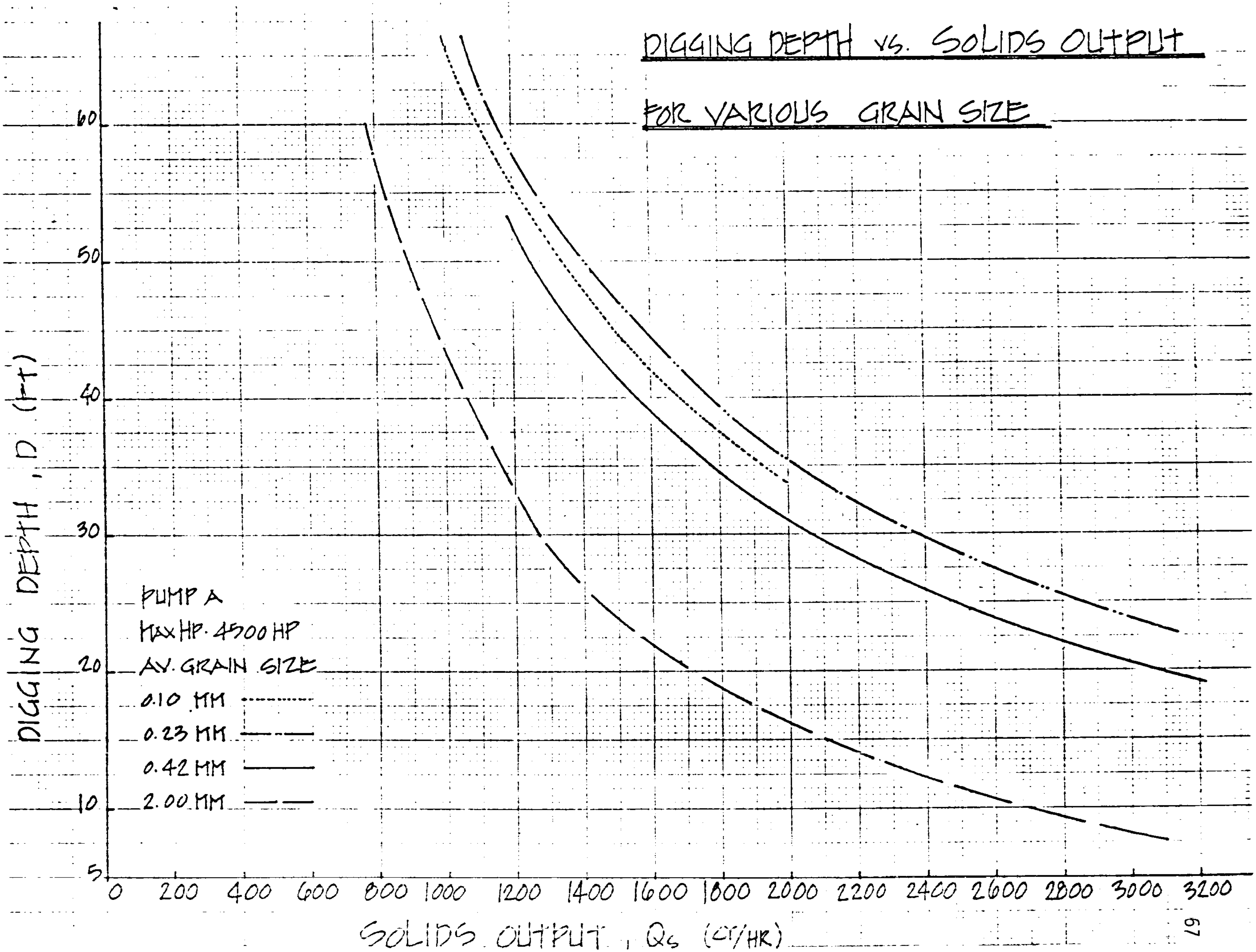


Fig. 18 EFFECTS OF SOIL MEDIAN GRAIN SIZE ON THE LARGE DREDGE CAVITATION LIMITATION

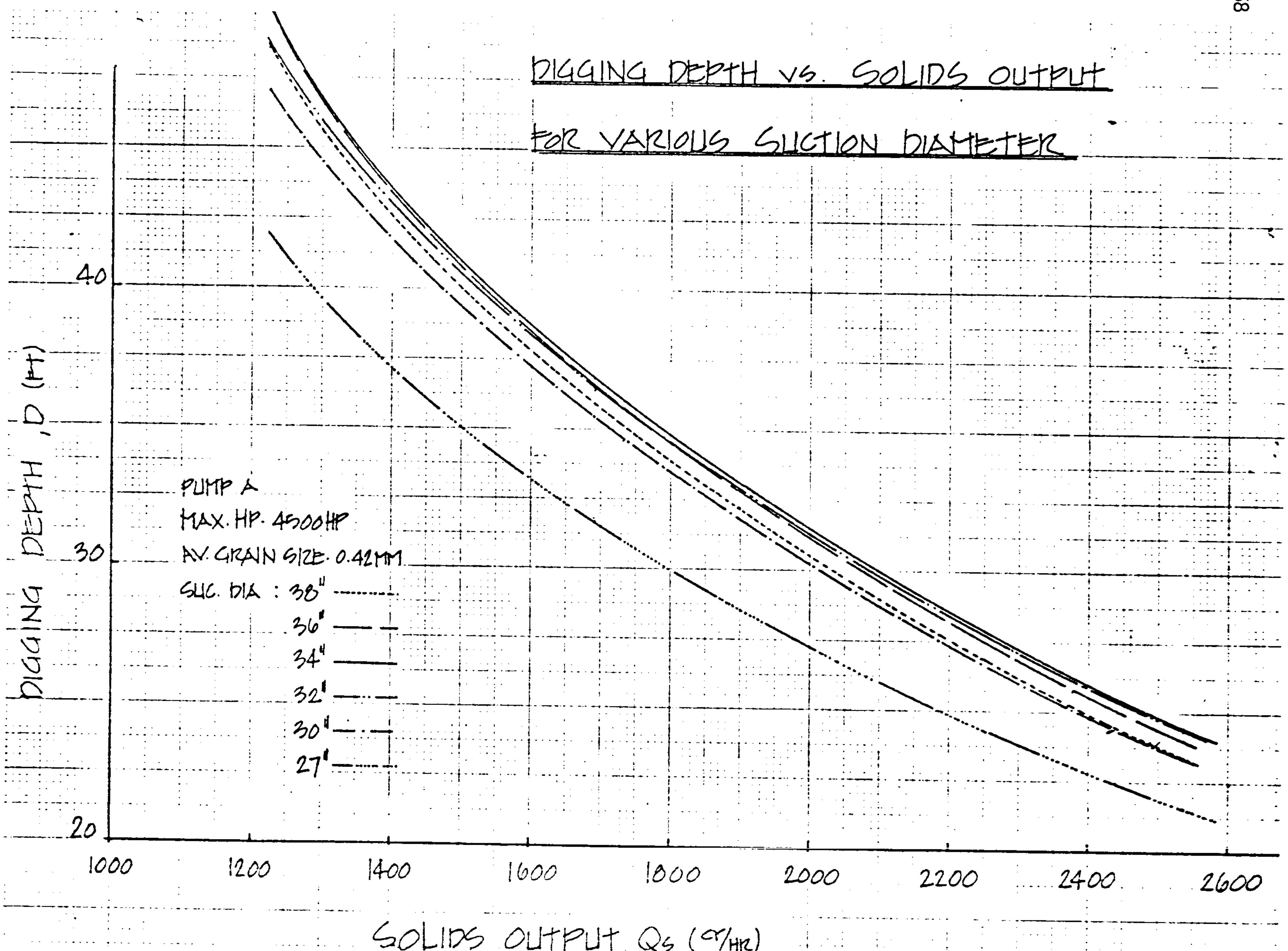


Fig. 19 EFFECTS OF SUCTION PIPE SIZE ON THE LARGE DREDGE CAVITATION LIMITATION

system discussed in this report was adapted (Fig. 1) employed a 34-inch diameter suction pipe. Here is one example where years of practical experience and modification have verified the results of the computer simulation model. Only a limited amount of field data was available to test the horsepower limitation and is discussed below.

VERIFICATION OF COMPUTER MODEL

Dredges must be instrumented to continuously measure slurry volumetric flowrate and slurry density such that required field data can be obtained to compare results described in this report with actual field results. Unfortunately, most hydraulic pipeline dredges do not use this equipment. Ways to overcome the problems inherent in any attempted field data collection program were discussed in a previously mentioned report[1].

One approximate method to overcome this problem is to employ measurements of the in situ volume of dredge material removed in the cut, below the water surface, over the period of time the dredge was operating. This solids output rate is defined as:

$$Q_s^* = \text{in situ solids output rate} = \frac{\text{volume in cut}}{\text{dredging time}} \qquad (7)$$

and is NOT the same as the solids output rate, Qs, used throughout this report and defined as:

$$Q_S = \text{actual solids output rate} = Q_T \cdot C_v \qquad (8)$$

where:

Q_T = total volumetric flowrate of pumped slurry, L^3/T

C_v = volume concentration of pumped slurry.

Eqn. 8 assumes the slip of the solid particles with respect to the fluid is very small or approximately zero which is a good approximation for velocities and concentrations employed in dredging practice. Consequently, the relationship between Q_s^* and Q_s must be determined in order for the in situ field data to be employed.

Relationship Between Q_s^* and Q_s for Various γi

Define the in situ, unit weight of wet, saturated soil beneath the water surface as γi. In Fig. 20, the approximate values of γi, for many types of dredge soils graded according to the Unified Soil

UNIT WEIGHTS OF DREDGED SOILS

	TYPICAL NAMES	UNIFIED SOIL SYMBOL	Max Dry Density, lb/ft³	Optimum Water Content %	Total Wet Density, lb/ft³	Average lb/ft³	REMARKS
GRAVELS	Well graded gravels	GW	119 ±	13.3 -	127.3	123	50% larger than #4 sieve (4.76 mm)
	Poorly graded gravels	GP	110 ±	12.4 -	117.7		
	Silty gravels	GM	114 ±	14.5 -	123.0		
	Clayey gravels	GC	115 ±	14.7 -	124.2		
SANDS	Well graded sands	SW	119 ± 5	13.3 ± 2.5	127.3 ± 6.6	124	
	Poorly graded sands	SP	110 ± 2	12.4 ± 1.0	117.7 ± 2.7		
	Silty sands	SM	114 ± 1	14.5 ± 0.4	123.0 ± 1.3		
	Silty-Clayey sands	SM-SC	119 ± 1	12.8 ± 0.5	127.0 ± 1.3		
	Clayey sands	SC	115 ± 1	14.7 ± 0.4	124.2 ± 1.3		
SILTS & CLAYS	Inorganic silts, sands	ML	103 ± 1	19.2 ± 0.7	115.0 ± 1.7	118	smaller than #200 (0.074 mm)
	Inorganic silt-clays	ML-CL	109 ± 2	16.8 ± 0.7	119.5 ± 2.4		
	Inorganic clays	CL	108 ± 1	17.3 ± 0.3	118.8 ± 2.7		
	Organic silts	OL	-			107	
	Inorganic, elastic silts	MH	82 ± 4	36.3 ± 3.2	104.7 ± 5.9		
	Inorganic, fat clays	CH	94 ± 2	25.5 ± 1.2	109.9 ± 2.8		
	High plasticity clays	OH	-				

Fig. 20 ESTIMATED IN SITU, SATURATED UNIT WEIGHTS (γ_i) OF DREDGED SOIL

Classification System, have been computed. Data for the maximum dry density and optimum water content were taken from field tests of over 1200 different soils by the USBR.[6] Fig. 20 should be used for estimating purposes only.

The basic relationship for volume concentration, C_v can be defined as:

$$C_v = \frac{S_m - S_w}{S_s - S_w} \tag{9}$$

where: S_m = specific gravity of slurry mixture

S_w = specific gravity of transporting water

= 1.00 fresh water

= 1.03 sea water

S_s = specific gravity of solids

2.65 for quartz, feldspar sands.

Define C_v^* as the in situ concentration by volume of the wet saturated soil beneath the water surface. Hence, C_v^* and Q_s^* are related. It can be shown that for fresh water (S_w = 1.0):

$$C_v^* = C_v \frac{S_s - 1}{\left(\frac{\gamma i}{\gamma w} - 1\right)} \tag{10}$$

The relationship between C_v^* and C_v is plotted in Fig. 21 for various representative values of γi. In the limit, as the in situ voids approaches zero, the ratio C_v/C_v^* approaches 1.0. This would be equivalent to an in situ unit weight of solid rock (S_s = 2.65) and γi = 165 lb/ft^3. From Fig. 21, values of the ratio C_v/C_v^* (slope of line) have been obtained for each γi of interest and plotted in Fig. 22. The plot is linear and the straight-line equation is

$$\frac{C_v}{C_v^*} = 0.00971\ \gamma i - 0.6059 \tag{11}$$

But, by definition

$$\frac{Q_s}{Q_s^*} = \frac{C_v}{C_v^*} \tag{12}$$

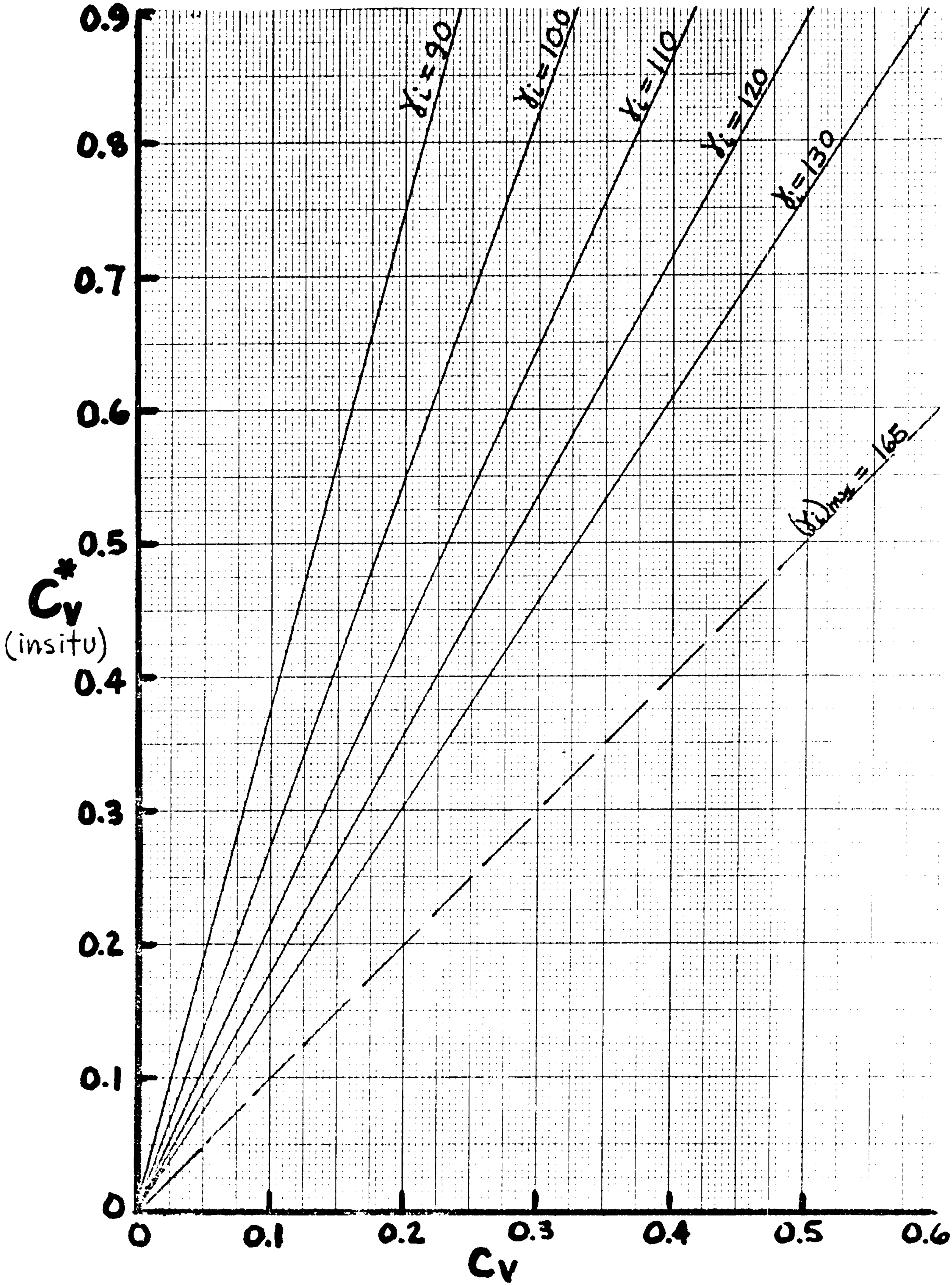


Fig. 21 RELATIONSHIP BETWEEN C_c^* AND C_v FOR VARIOUS γ_i

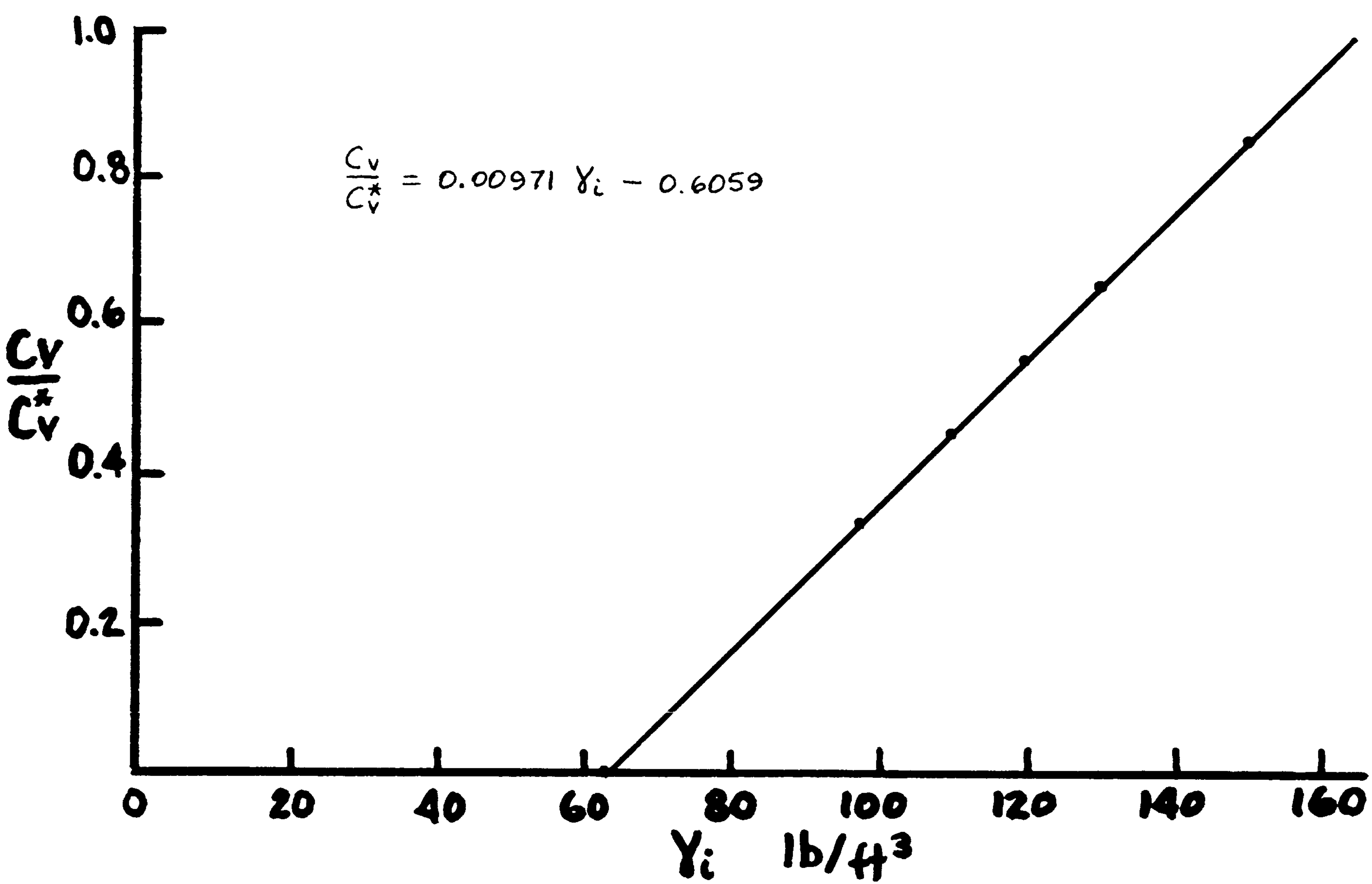


Fig. 22 RELATIONSHIP BETWEEN THE RATIO C_v/C_v^* and γ_i

hence, for approximation purposes

$$\frac{Q_s}{Q_s^*} \dot{\approx} \frac{\gamma i}{100} - 0.6 \qquad (13)$$

Thus, for a given in situ soil unit weight i (Fig. 20), the desired relation between Q_s and Q_s^* is obtained. For the size sands used in this report, $\gamma i \dot{\approx} 124$ lb/ft^3 so that

$$Q_s \dot{\approx} 0.64\ Q_s^* \quad \text{(Estimation only)} \qquad (14)$$

What this means is that because of the water present in the voids if the in situ volume of dredge-cut removed, the actual solids output rate measured by pipeline slurry meter must always be <u>less</u> than that expressed by classic dredging definition for production, which is volume of material removed over time dredged.

Another complication enters in, since during the dredging cycle the concentration varies continuously from maximum (approximately $S_m \dot{\approx}$ 1.5 to 1.6) to pure water (S_m = 1.0). Hence, some estimate of the dredging efficiency must be made (usually 50% is assumed for lack of sufficient data) in order to convert average dredge output back to an estimate for maximum solids production assuming no periods of zero solids concentration (pure water) were required. This would be equivalent to dividing the right side of Eqn. 14 by the dredging efficiency (50-100%). What happens therefore with these two corrections is that they <u>tend</u> to cancel each other. This means that one can roughly compare the solids output rates Q_s directly with in situ production estimates Q_s^*, recognizing the limitations and approximations built into these direct comparisons.

<u>Sample Field Data</u>

Some sample field data necessary to compute Q_s^* was obtained from the Gahagen Dredging Co. on their Woodmere project and is tabulated in Fig. 23. The material was silty-clayey sand and the estimate of Q_s is also shown in Fig. 23 based on Eqn. 13. However, assuming a dredging efficiency such that Q_s^* and Q_s are approximately equal, the field data are plotted in Fig. 24 which is identical to that previously discussed in Fig. 10. The large dredge system employed, as an example in this report, (Fig. 1),

WOODMERE

WITHOUT BOOSTER

DAYS	LINE	OPER. HRS.	C.Y.	Qs* C.Y./HR.	Qs C.Y./HR.	DATE
1	7500-8000	19:35	11,070	1190	700	1-28
3	8000-8500	35:40	41,700	1169	688	1-13,29,31
4	8500-9000	74:50	94,000	1254	738	1-14 2-1,4,5
6	9000-9500	82:15	101,550	1234	726	1-15 2-2,3,6,7,13
6	9500-10000	110:25	109,150	989	581	1-17,18,26 2-8,12,14
2	10000-10500	27:45	20,750	748	440	1-19 2-21
3	10500-11000	65:25	41,600	636	374	1-20,21 2-9
7	11000-11500	127:50	99,100	775	456	1-22,24,25,27 2-10,15,16
2	11500-12000	45:10	36,350	804	473	2-11,17
4	12000-12500	75:10	60,250	802	472	2-18,19
1	12500-13000	23:20	16,000	686	403	2-21
3	13000-13500	57:35	44,300	769	452	2-24,25,26

Fig. 23 EXAMPLE FIELD DATA FOR WOODMERE PROJECT (COURTEST GAHAGEN DREDGING CO.)

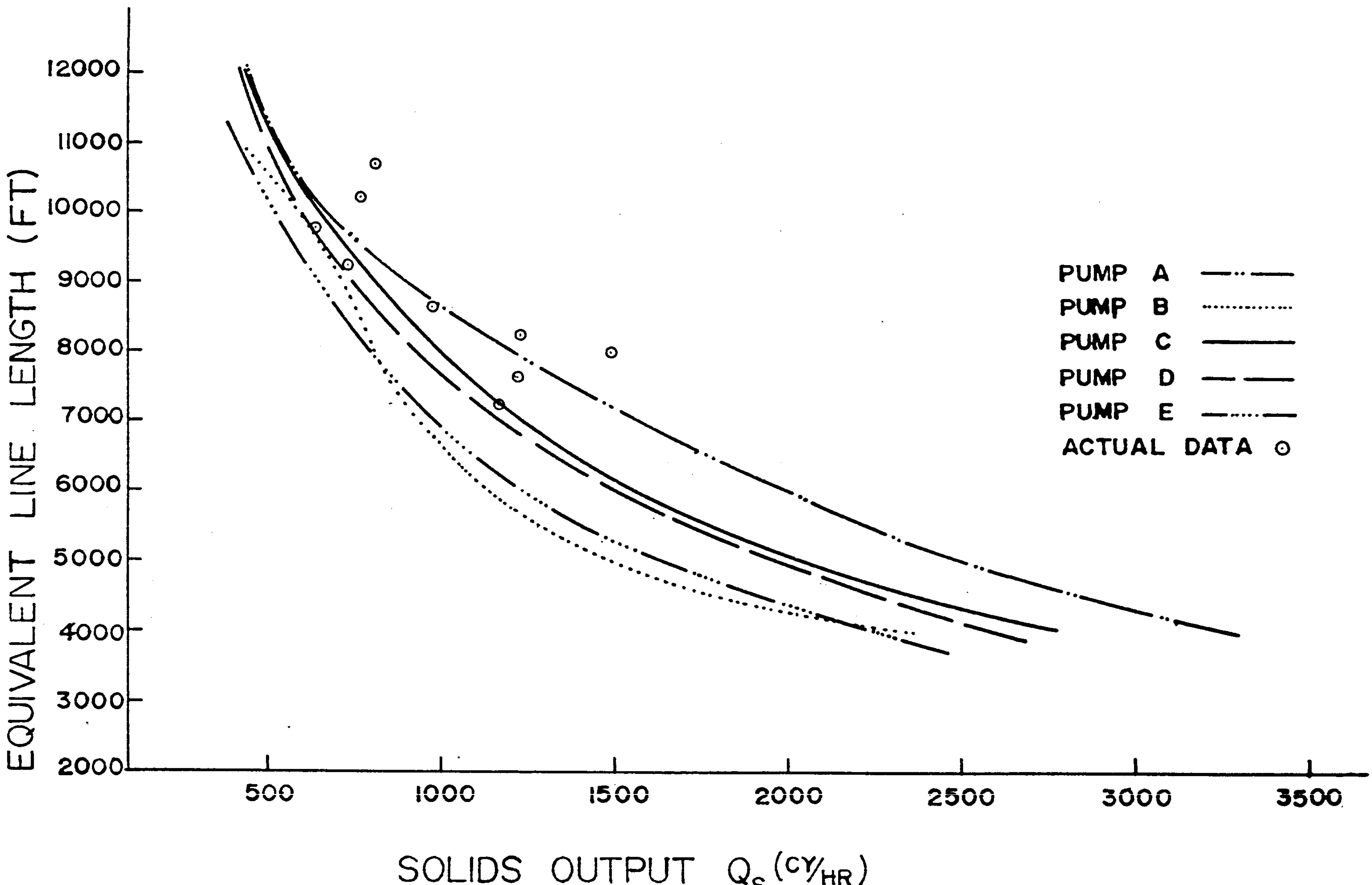


Fig. 24 COMPARISON BETWEEN COMPUTER RESULTS AND FIELD DATA FOR LARGE DREDGE--HORSEPOWER LIMITATION ONLY

was modeled after the dredge used by Gahagen for the Woodmere project. Unfortunately, no dredge-pump test information was available for this 27-inch Gahagen dredge. Therefore the plotted field data can only be qualitatively used as verification of the computer model. However, much of the data fell close to the predicted solids output curves and the results are encouraging.

SUMMARY AND CONCLUSIONS

We have briefly reviewed the elements of an equation-based model of hydraulic dredging systems which correctly demonstrates the effects of line length, digging depth and grain size on solids output of the dredge. Limited field data "agreed" with the model results.

A test program was devised and is being carried out to investigate the relative influence on solids output of the many variables involved in the dredging process. One of the key tests involved the effects of five (5) different dredge pump designs on solids output.

Based on the pump performance curves available and devised for these tests, and the use of the computer model described herein we have drawn the following conclusions:

(1) The "one-dredge pump is as-good-as-another" (hydraulically) hypothesis must be rejected. Substantial differences in production (50%±) were demonstrated for both the horsepower (line-length) and cavitation (digging depth) solids output limiting criteria. (2) The model dredge program can systematically and economically study the effects of variables such as horsepower available, suction-pipe size, particle size, and others not considered herein.

This work is continuing, in order to examine additional variables such as booster pumps (suction & discharge), pump location, terminal elevation, discharge pipe size, etc.

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MAINTENANCE DREDGING IN THE NEW ORLEANS DISTRICT US ARMY CORPS OF ENGINEERS

By

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New Orleans District
U.S. Army Corps of Engineers

INTRODUCTION

Figure 1 shows the New Orleans District of the Corps of Engineers. This district covers an area of 47,000 square miles. It includes most of the State of Louisiana and small portions of the States of Mississippi, Arkansas and Texas.

At the present time the New Orleans District is responsible for operating and maintaining 42 authorized navigation projects including the navigational features of 4 authorized flood control projects. There are approximately 2,800 miles of navigable waterways involved, of which 405 miles are deep draft waterways.

The number of miles of waterways dredged each year amounts to 110 deep draft and 60 shallow draft. The deep draft channels have authorized depths ranging from 30 to 42 feet; the shallow draft from 5 to 20 feet.

For the 5-year period 1968 through 1972, maintenance dredging averaged 61.3 million cubic yards per year. We think of this quantity as representative of a normal year. It amounts to approximately 25% of the maintenance dredging program for the entire Corps of Engineers. 1973 and 1974 were by no means normal years in terms of maintenance dredging requirements in this district. The flood of 1973 and the near-flood of 1974 caused marked increases in the dredging requirements in the Mississippi River, not only in this district but also in some of the other districts upstream. We will discuss the 1973 and 1974 dredging requirements later.

In a normal year, execution of the New Orleans District's dredging program involves the use of one or two Government-owned hopper dredges, one Government-owned dustpan dredge, one or two contract clamshell bucket dredges and 15 to 20 contract cutterhead pipeline dredges. No government-owned cutterhead dredges are used in the New Orleans District.

Fig. 1 New Orleans District of the Corps of Engineers.

DREDGING ACTIVITIES

Most of the dredging takes place in the southern portion of Louisiana. Figure 2 shows the principal navigation projects involved. There are three deep-draft projects. The eastern-most is the Mississippi River-Gulf Outlet, with a depth of 36 feet and a width of 500 feet, except for the gulf entrance portion which is 38 by 600 feet. West of the Mississippi River-Gulf Outlet is the Mississippi River with an authorized depth of 40 feet from Baton Rouge to the Gulf of Mexico, and authorized widths from 500 feet to 1000 feet. The third ship channel is Calcasieu River and Pass, near the western boundary of the district. It serves the Port of Lake Charles and has authorized dimensions of 40 by 400 feet from Lake Charles to the coastline and 42 by 800 feet from the coastline to the 42-foot contour in the Gulf of Mexico.

The most important of the shallow draft channels is the Gulf Intracoastal Waterway (12 feet deep by 125 feet wide). Its main stem is parallel to the Gulf Coast and is part of the larger Intracoastal Waterway project which stretches from Apalachee Bay, Florida, to Brownsville, Texas. The Morgan City-Port Allen Alternate Route of the Gulf Intracoastal Waterway (12 by 125 feet) connects the Mississippi River at Baton Rouge to the main stem of the GIWW at Morgan City.

Some of the other shallow draft channels are: (a) the Barataria Bay Waterway, 12 feet by 125 feet, which runs from the Intracoastal Waterway at Barataria to the Gulf of Mexico; (b) the Houma Navigation Canal, 15 feet by 150 feet from the Intracoastal Waterway at the City of Houma to the Gulf of Mexico; (c) Atchafalaya River, 12 by 125 feet, from Old River to Morgan City and 20 by 400 feet from Morgan City to the Gulf of Mexico; and (d) Freshwater Bayou, 12 by 125 feet, from the Gulf Intracoastal Waterway at Intracoastal City to the Gulf.

There are a number of smaller projects which I will not enumerate at this time.

The navigation channel of the Mississippi River from Cairo, Illinois, to Baton Rouge, La., has authorized project dimensions of 12 feet by 300 feet. However, only 9 feet by 300 feet is being maintained until control works for stabilizing the channel are completed. The river enters the New Orleans District near Artonish, Miss., approximately 90 river miles above Baton Rouge. Within this 90-mile reach there are 8 crossings, i.e.,

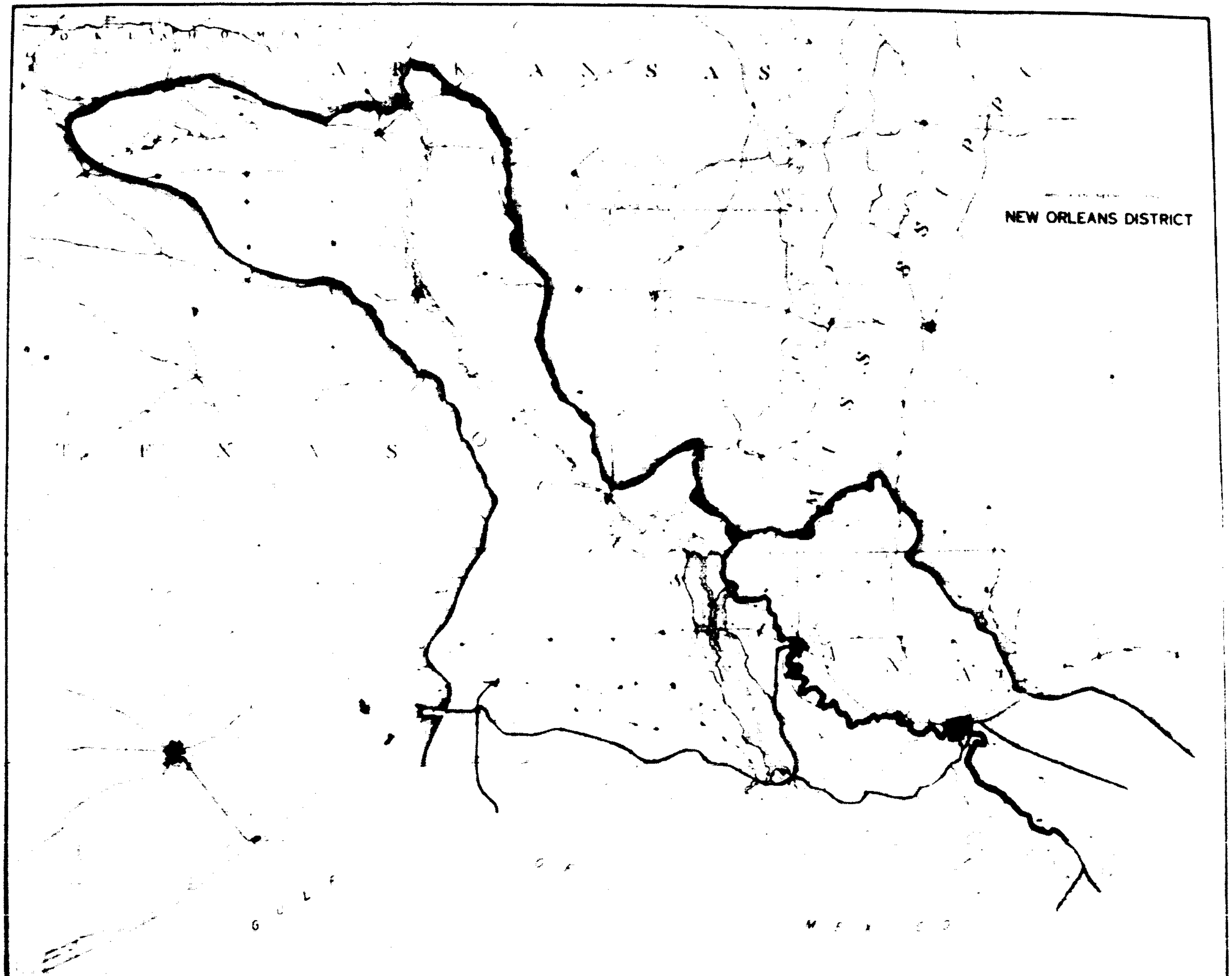

Fig. 2. Principal navigation projects.

places where the thalweg crosses from one side of the river to the other. Shoaling tends to occur at the mid-point of these crossings. Each crossing has its own shoaling characteristics. Usually only one or two of these crossings requires dredging in any one year. This dredging is accomplished by the US Dustpan Dredge JADWIN and amounts to approximately 1.5 million cubic yards annually. Spoil is placed in shallow water as far as practicable from the dredged channel.

BATON ROUGE TO NEW ORLEANS CHANNEL

The largest and most important, and I think the most interesting, navigation project in the New Orleans District is the Mississippi River from Baton Rouge to the Gulf of Mexico (Figure 3). It provides deep draft access to the Port of New Orleans, the third largest in the world and second largest in the United States, and to Baton Rouge, the seventh largest port in the nation.

This project provides for a channel 40 feet deep by 500 feet wide from Baton Rouge to New Orleans; in New Orleans Harbor a channel 35-feet-deep by 1500-feet-wide measured from a line 100-feet-riverward from the face of the east bank wharves, and a 40-foot by 500-foot channel within the 1500-foot channel. From New Orleans to Head of Passes a channel 40 feet by 1000 feet is provided. In Southwest Pass a channel 40 feet by 800 feet is provided with a 40-by 600-foot channel at the lower end of the jetties and over the bar into the Gulf of Mexico. In South Pass a 30-by 450-foot channel is maintained, with a 30-by 600-foot channel over the bar.

All of the depths above New Orleans are referred to average low water, while those in New Orleans Harbor and downstream to the Gulf of Mexico are referred to Mean Low Gulf level.

DEEP WATER CROSSINGS

Figure 4 shows the locations and names of the nine deepwater crossings between Baton Rouge and New Orleans. Seven of these crossings require annual maintenance. This work is done by the Dustpan Dredge JADWIN. Approximately 5 million cubic yards are removed annually. Spoil is disposed of in the shallow water, outside of the dredged channel.

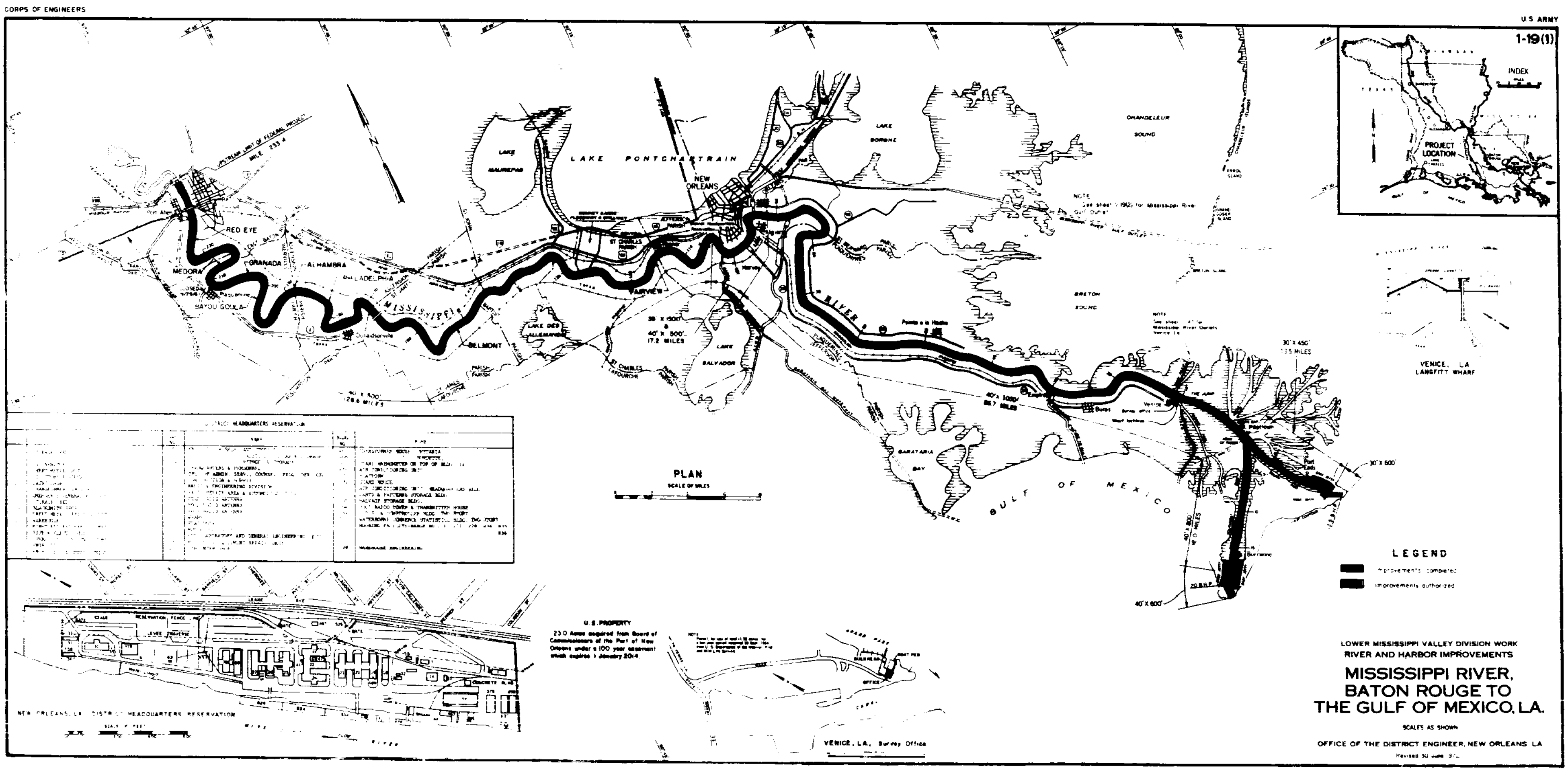

Fig. 3. Mississippi River from Baton Rouge to the Gulf of Mexico.

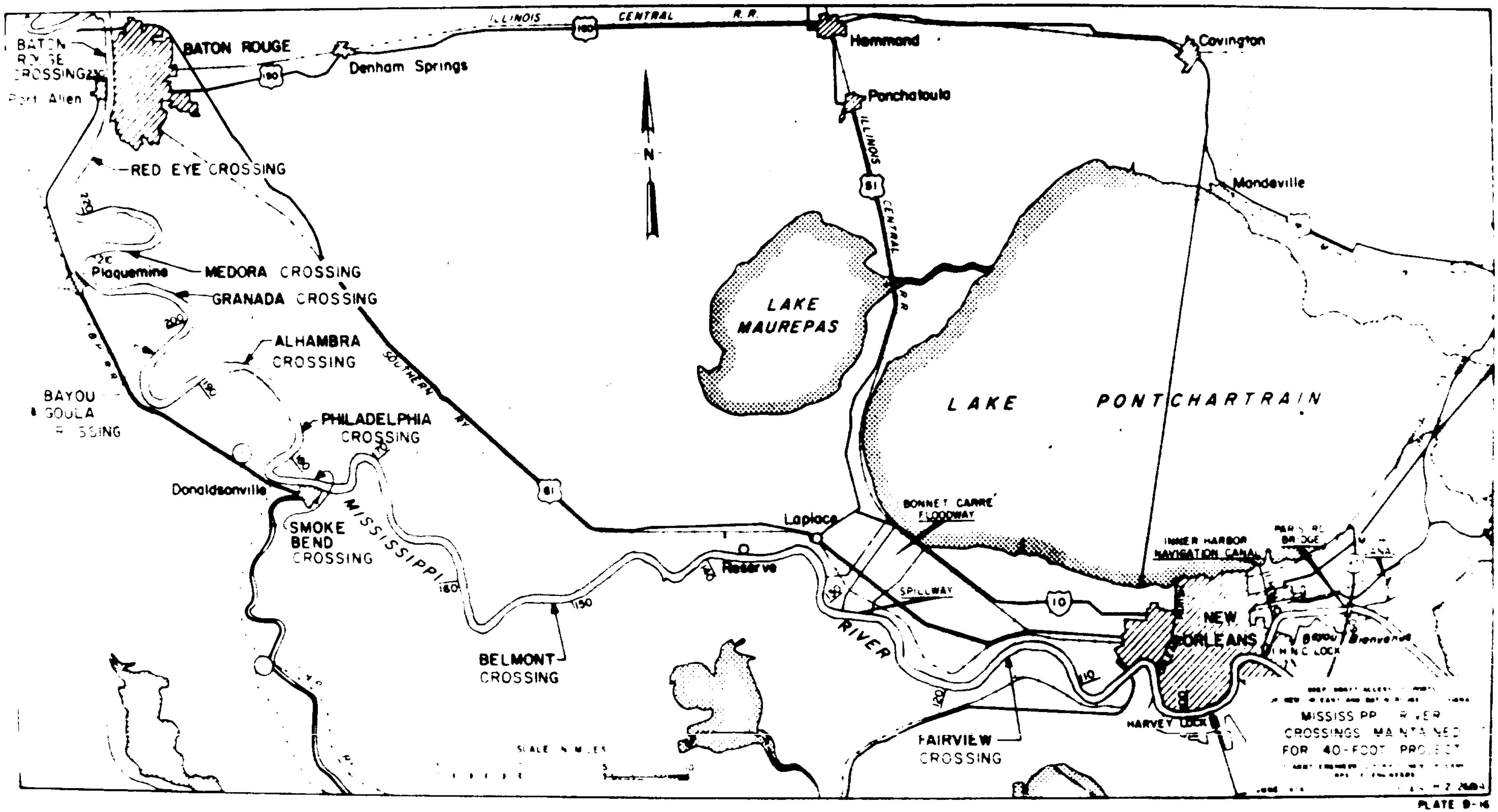

Fig. 4. Locations of the nine deepwater crossings between Baton Rouge & New Orleans.

DUSTPAN DREDGE JADWIN

For those of you who may not be familiar with the dustpan type dredge, here are two pictures of the JADWIN (Figures 5 and 6).

NEW ORLEANS HARBOR

Figure 7 is a map of the New Orleans Harbor area. Dredging is required annually. It usually begins in mid-April and is finished in early October.

The Board of Commissioners of the Port of New Orleans, generally referred to as the Dock Board, is responsible for maintenance of the 100-foot-wide areas adjacent to the east bank wharves. The Dock Board's dredging amounts to about 1.5 million cubic yards per year.

As mentioned earlier, the Government is responsible for maintenance of a 1500-foot-wide channel riverward to the 100-foot area maintained by the Dock Board. Because of the river's natural depth, it is necessary to dredge only 10 to 15% of this 1500-foot width to provide the required 35-foot depth.

The 40- by 500-foot channel which lies within the 1500-foot channel is naturally deep over its entire width and length and thus requires no dredging. The Government's harbor dredging amounts to about 2.2 million cubic yards per year. Cutterhead dredges (size range 20" to 24") are utilized for harbor maintenance. Spoil is placed in deep water in the river.

Figure 8 shows a dredge working in the harbor. Downtown New Orleans is in the background.

From the lower limit of New Orleans harbor, downstream to Head of Passes, a distance of approximately 87 river-miles, the natural cross section of the river exceeds the 40- by 1000-foot project requirement. No dredging is required in this reach; a break for the taxpayer.

MISSISSIPPI DELTA

Figure 9 is a map of the Mississippi River delta with its numerous distributary channels. Two of these distributaries, South Pass and Southwest Pass, are part of the deep draft navigation project which extends from Baton Rouge to the Gulf of Mexico. These two passes require annual maintenance. No dredging is done in any of the other passes since they are not authorized navigation projects. The junction of Soughe and Southwest Passes and the main stem of the river is called Head of Passes.

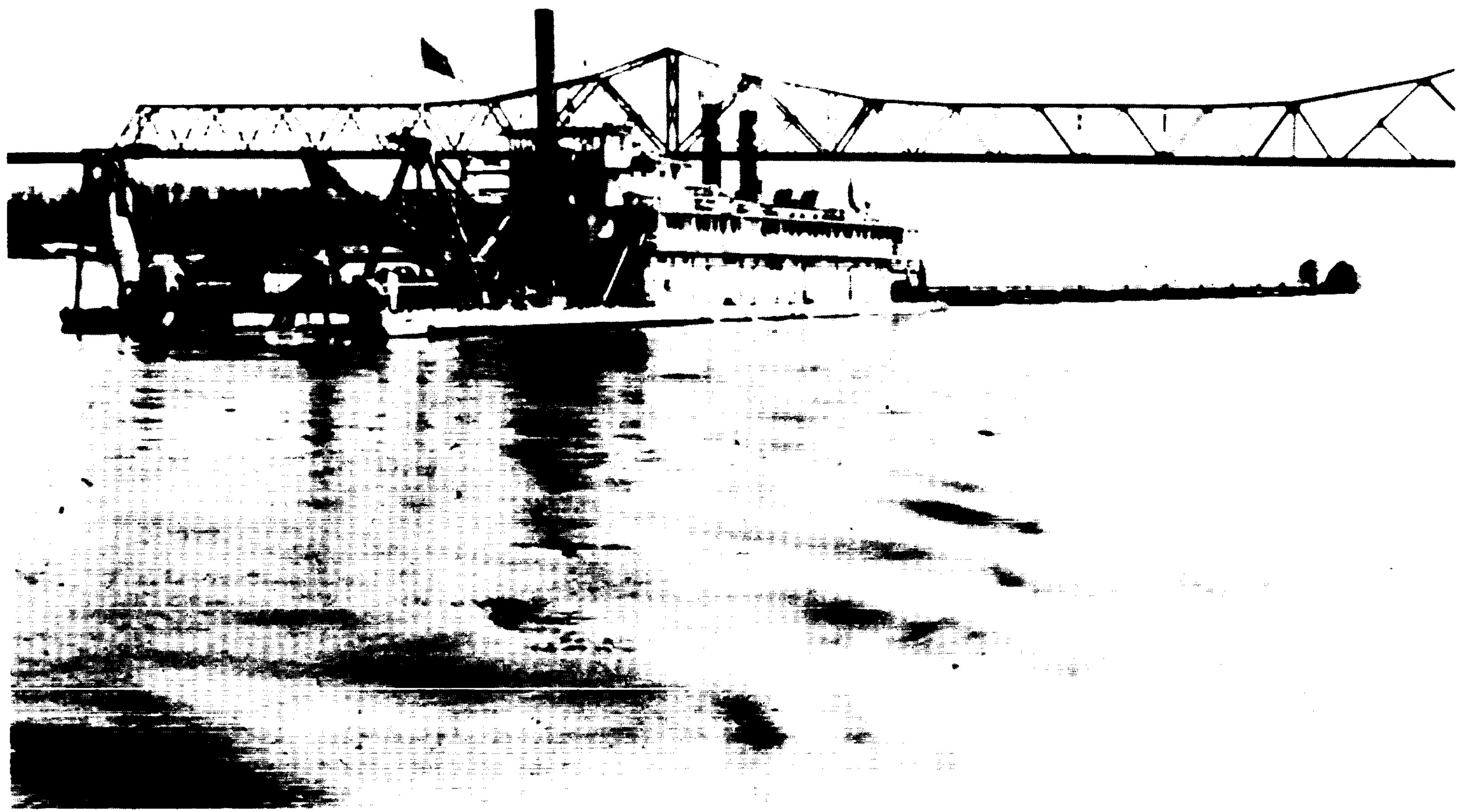

Fig. 5. JADWIN - A dustpan type dredge.

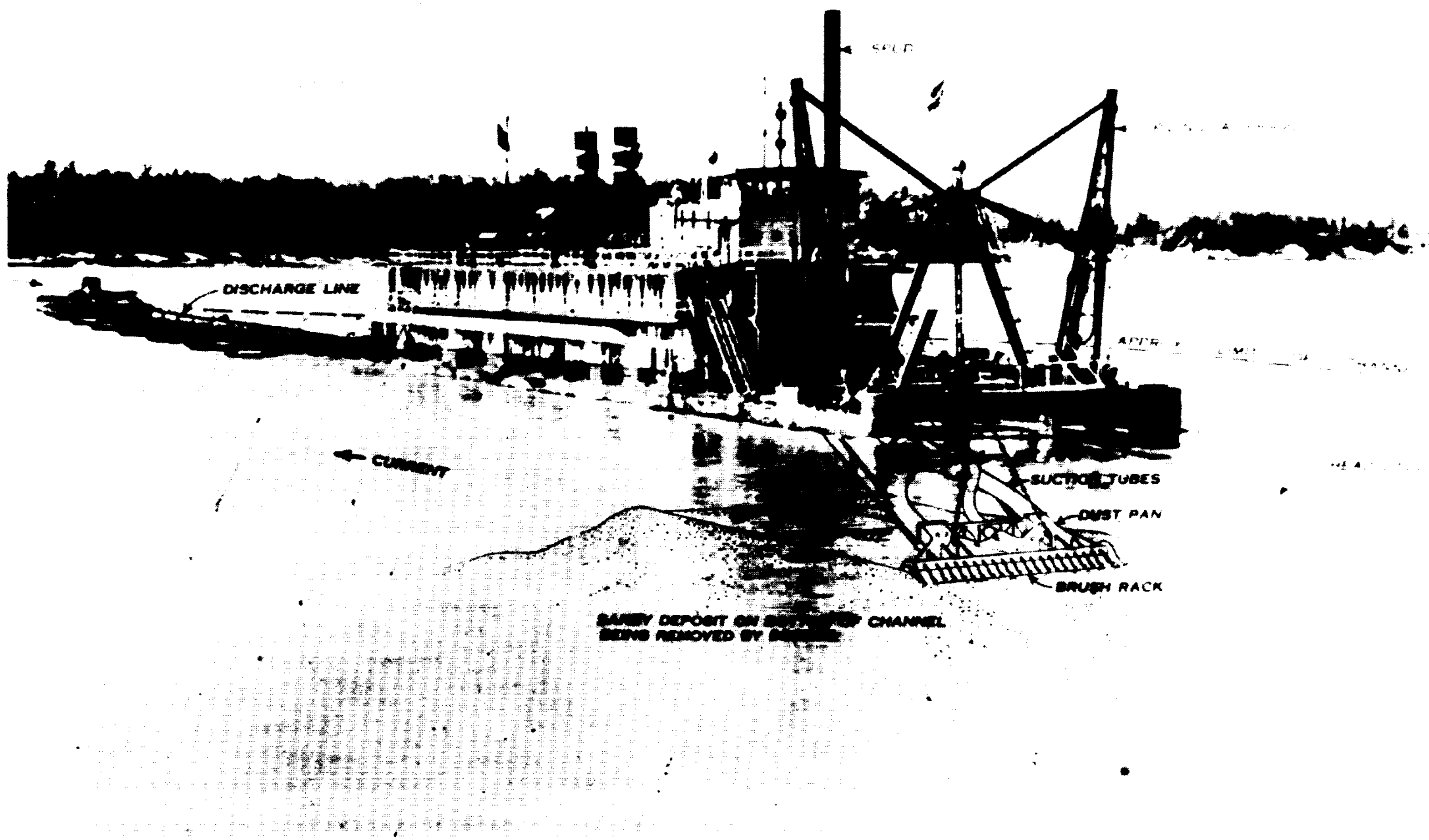

Fig. 6. JADWIN.

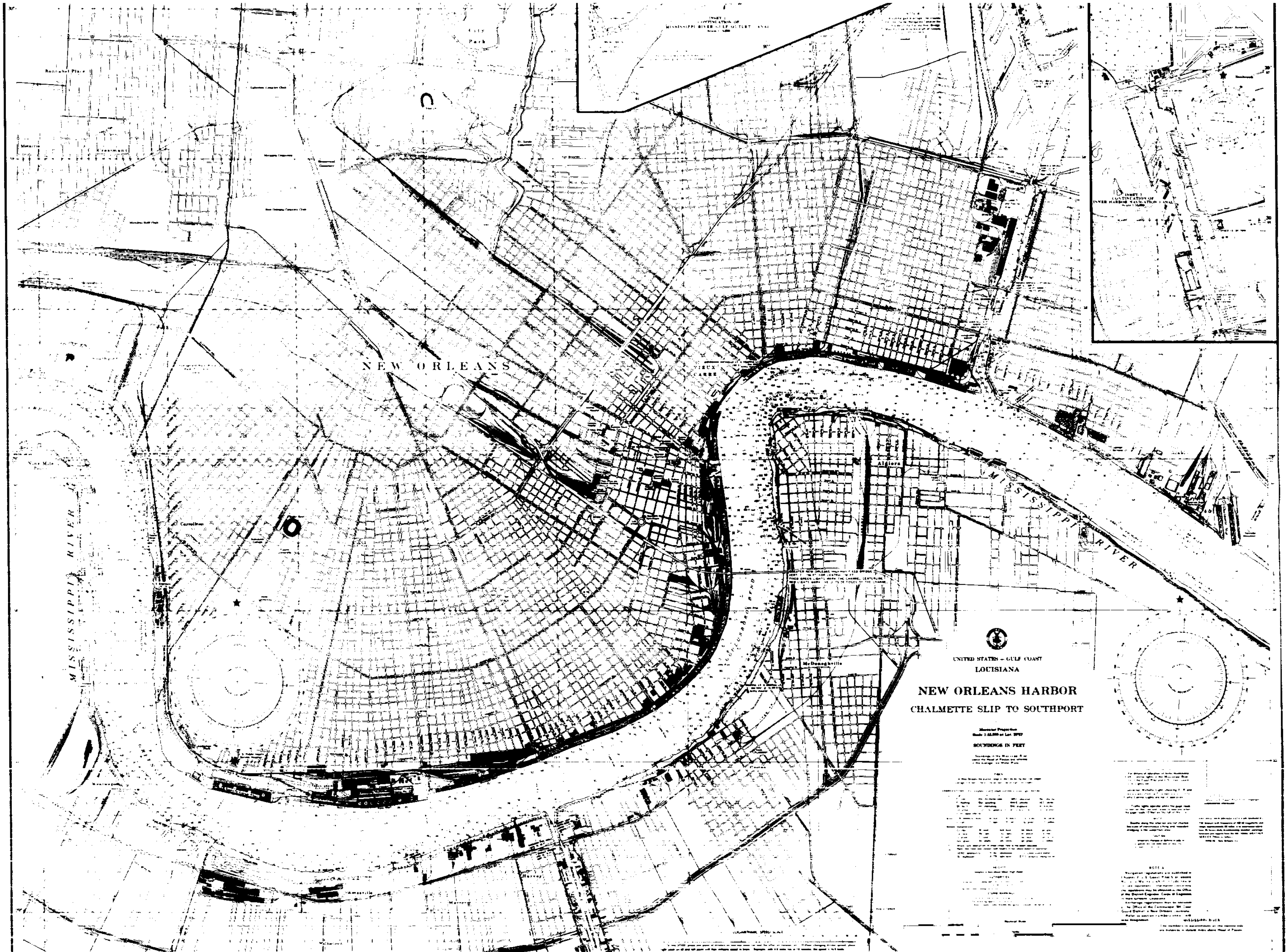

Fig. 7. New Orleans Harbor.

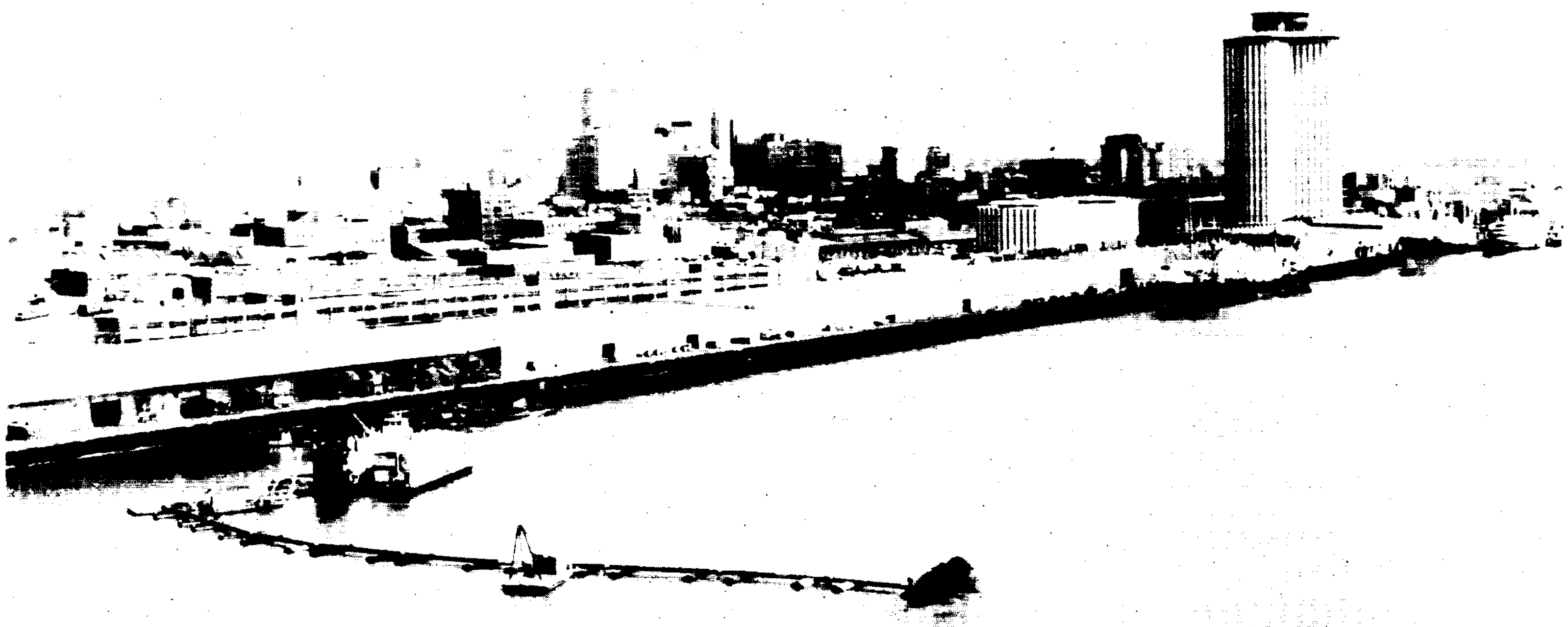

Fig. 8. Dredge working in New Orleans Harbor.

Fig. 9. Mississippi River Delta.

Mileages on the Lower Mississippi River (Cairo, Ill. to Gulf of Mexico) are measured from Head of passes. The maze of canals on both sides of Southwest Pass are for access to oil wells. One of the largest oil fields in the State underlies this general area.

Systems of jetties help to stabilize the channels at the lower end of South and Southwest Passes. The South Pass east jetty and Southwest Pass west jetty are of concrete construction buttressed with stone on each side; the other two jetties are constructed of quarry stone in sizes up to 10 tons (Figure 10). Headland structures protect the land points between Pass a Loutre and South Pass, and between South Pass and Southwest Pass. These are timber pile crib dikes filled with stone, constructed on willow mats. Permeable spur dikes of timber pile construction extend from the banks of Southwest Pass, perpendicular to the channel centerline. These dikes construct the channel and build the banks thus keeping flow velocities in the channel high enough to minimize shoaling. Outlets located at strategic points along the banks of South and Southwest Passes permit the flow of sediment-carrying river water out of the passes to nourish the banks.

Maintenance dredging in Southwest Pass between Head of Passes and mile 18.8 below Head of Passes (BHP) is accomplished annually with contract cutterhaed dredges. Approximately 6.0 million cubic yards are removed annually and placed in diked areas on either bank. Two dredges (24" to 30") are employed; they work from June to October. Most of the spoil areas are now full, or nearly so. Diking costs are high. Action has been initiated to acquire the much needed additional areas. The presence of the numerous oil wells will complicate this acquisition and restrict the use of some areas.

Figure 11 is a view looking downstream over Southwest Pass. The dredge PONTCHARTRAIN is in the foreground, about 9 miles BHP. Note the narrow spoil areas and the oil-field canals and slips.

The Hopper Dredge LANGFITT, (Figure 12), maintains the channel between mile 18.8 BHP and the lower end of the jetties, mile 20.2 BHP, and through the bar in the Gulf. Early in the season when flows are high, the LANGFITT dredges by means of agitation; however, toward the end of the season when velocities are low, the dredge-and-haul method is used

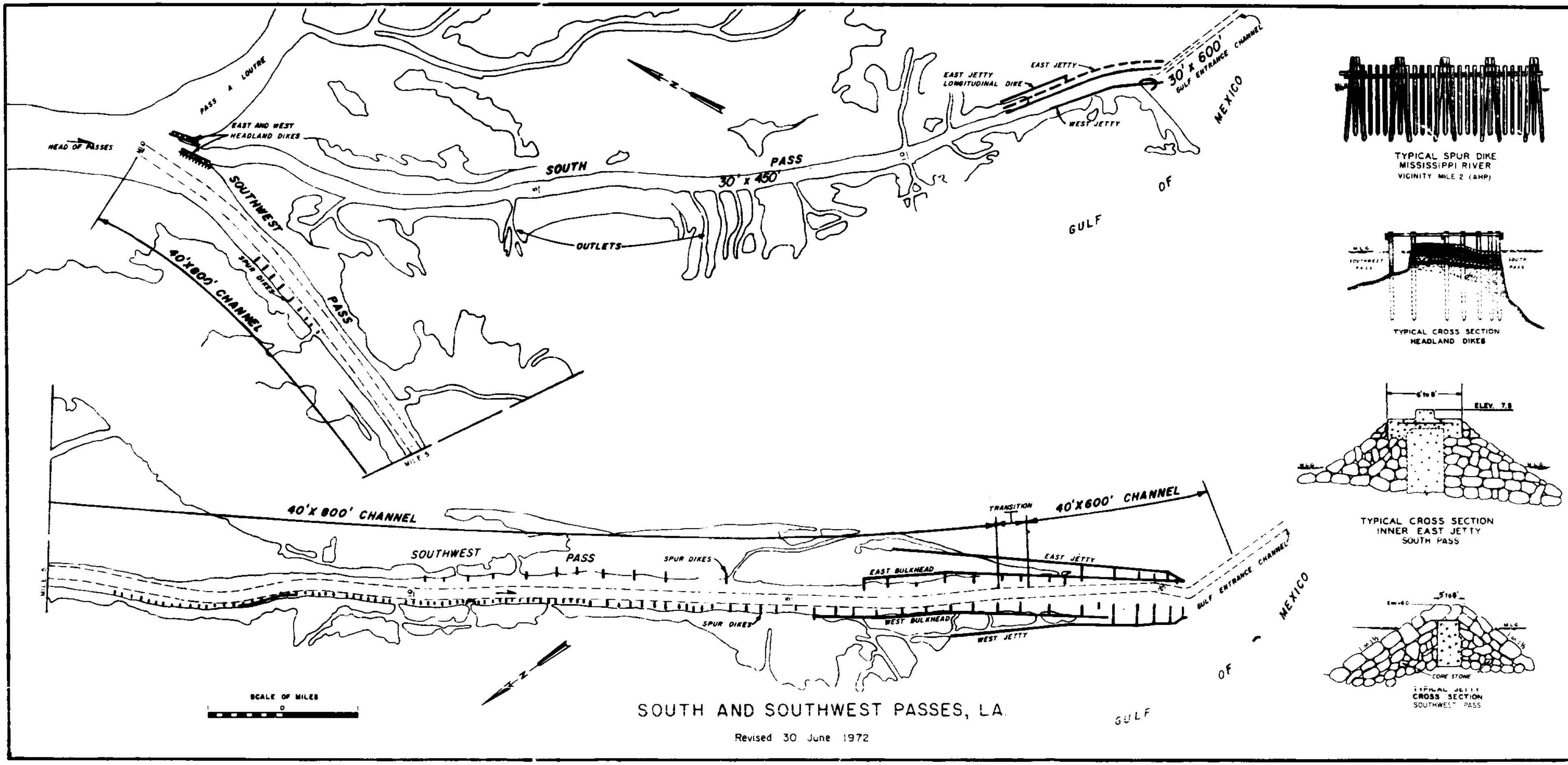

Fig. 10. Jetty construction in South & Southwest passes.

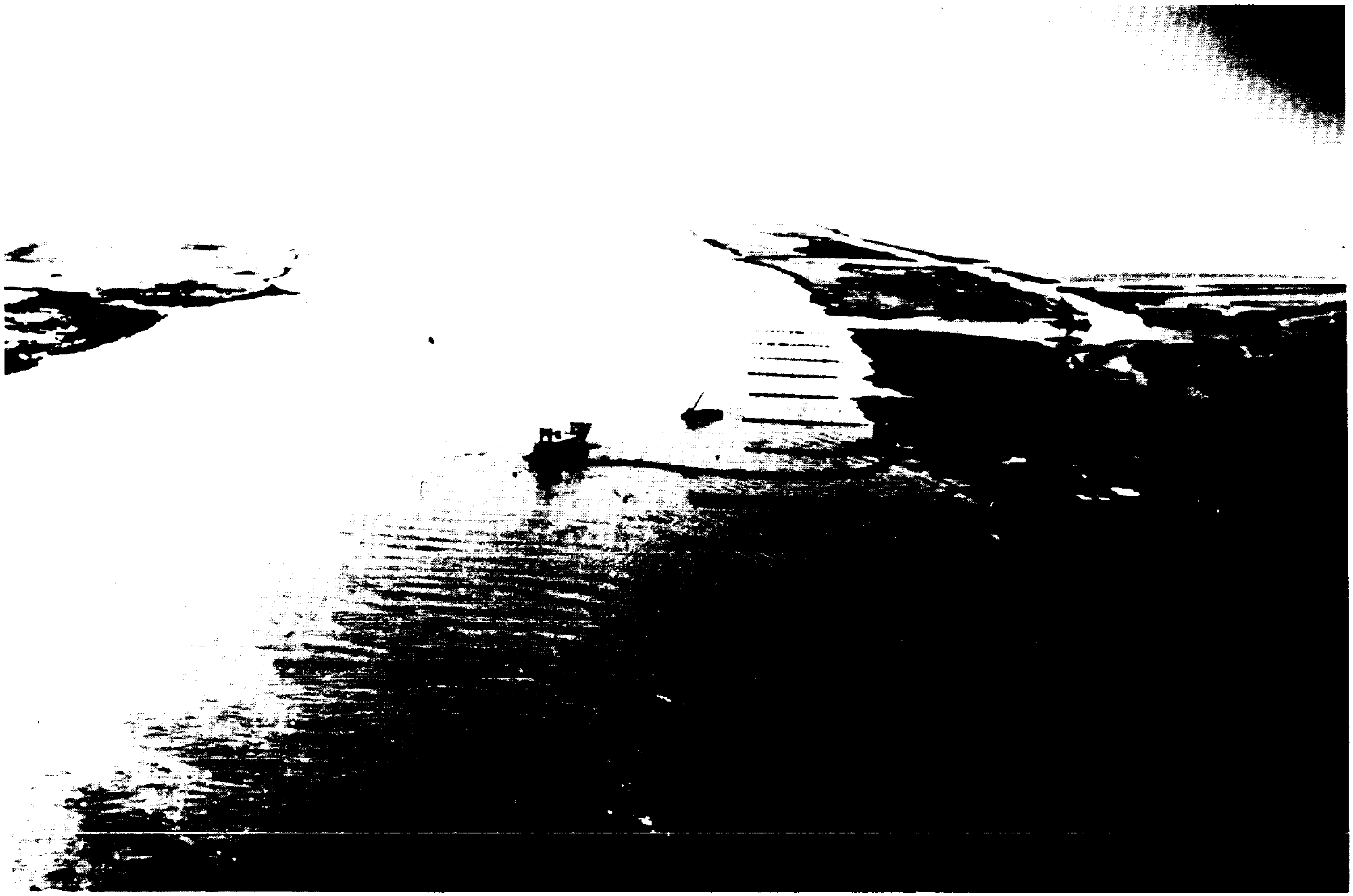

Fig. 11. The dredge PONTCHARTRAIN looking downstream over Southwest Pass.

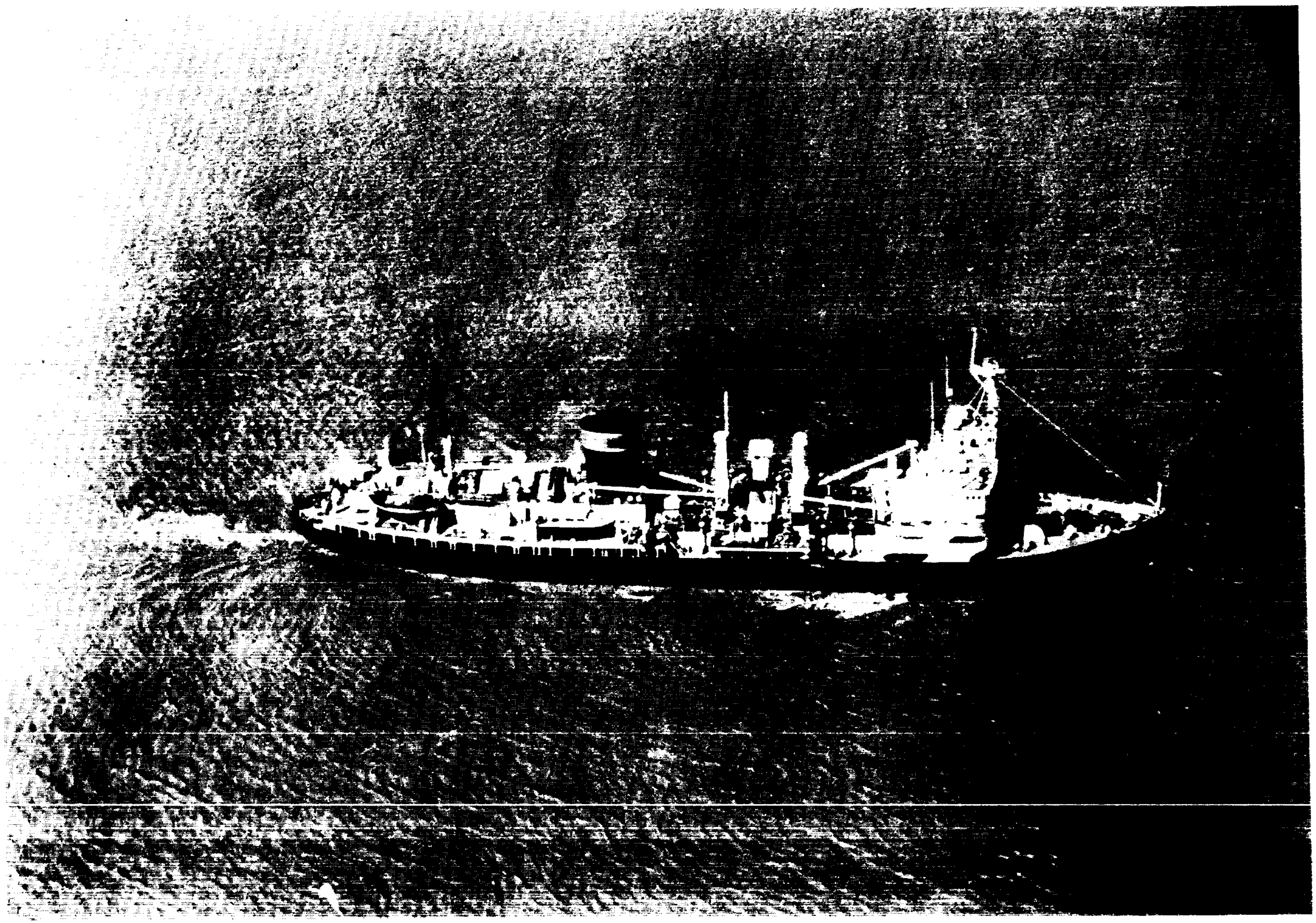

Fig. 12. The Hopper Dredge LANGFITT.

and spoil is dumped in deep water in the Gulf of Mexico. The LANGFITT removes about 5.5 million cubic yards of material from Southwest Pass annually. This work usually begins in February and lasts until June or July.

The LANGFITT has a hopper capacity of 3,000 cubic yards; two pumps with 30-inch intake and 28-inch discharge, each pump directly driven by a 1,150 HP electric motor; there are two 3,000 HP electric propulsion motors. Hopper dredges are ships in the true sense of the word. They are built in accordance with American Bureau of Shipping and Coast Guard requirements. Their officers are licensed by the Coast Guard the same as officers of other sea-going ships. They are capable of going anywhere in the world. Hopper dredges must be highly maneuverable. They are often required to turn around in channels only a few feet wider than their own length. This capability is provided by twin propellers and rudders, and in some cases by a bow thruster.

Maintenance dredging in South Pass is performed with a cutterhead dredge. The South Pass bar channel is maintained by the dredge LANGFITT. The cutterhead removes approximately 850,000 cubic yards and the LANGFITT about 1.6 million cubic yards.

Maintenance of the navigation project from Baton Rouge to the Gulf of Mexico, amounts to approximately 20.7 million cubic yards per year, or about 34% of the District's total maintenance dredging program.

THE MISSISSIPPI RIVER GULF OUTLET

The Mississippi River-Gulf Outlet has a length of about 76 miles; 43 miles of land cut, 23 miles through Breton Sound, and 9.5 miles from Breton Island to deep water in the Gulf of Mexico (Figure 13). In the land cut, dredge spoil is placed in diked areas along the right descending side of the channel. Material dredged from the Breton Sound and gulf entrance reaches is disposed of in open water on the right descending side of the channel, 3,000 feet from the channel centerline. When a hopper dredge works the gulf entrance, it dumps in deep water on the right descending side. Annual dredging of this project amounts to 15.5 million cubic yards, about 25% of the District's total.

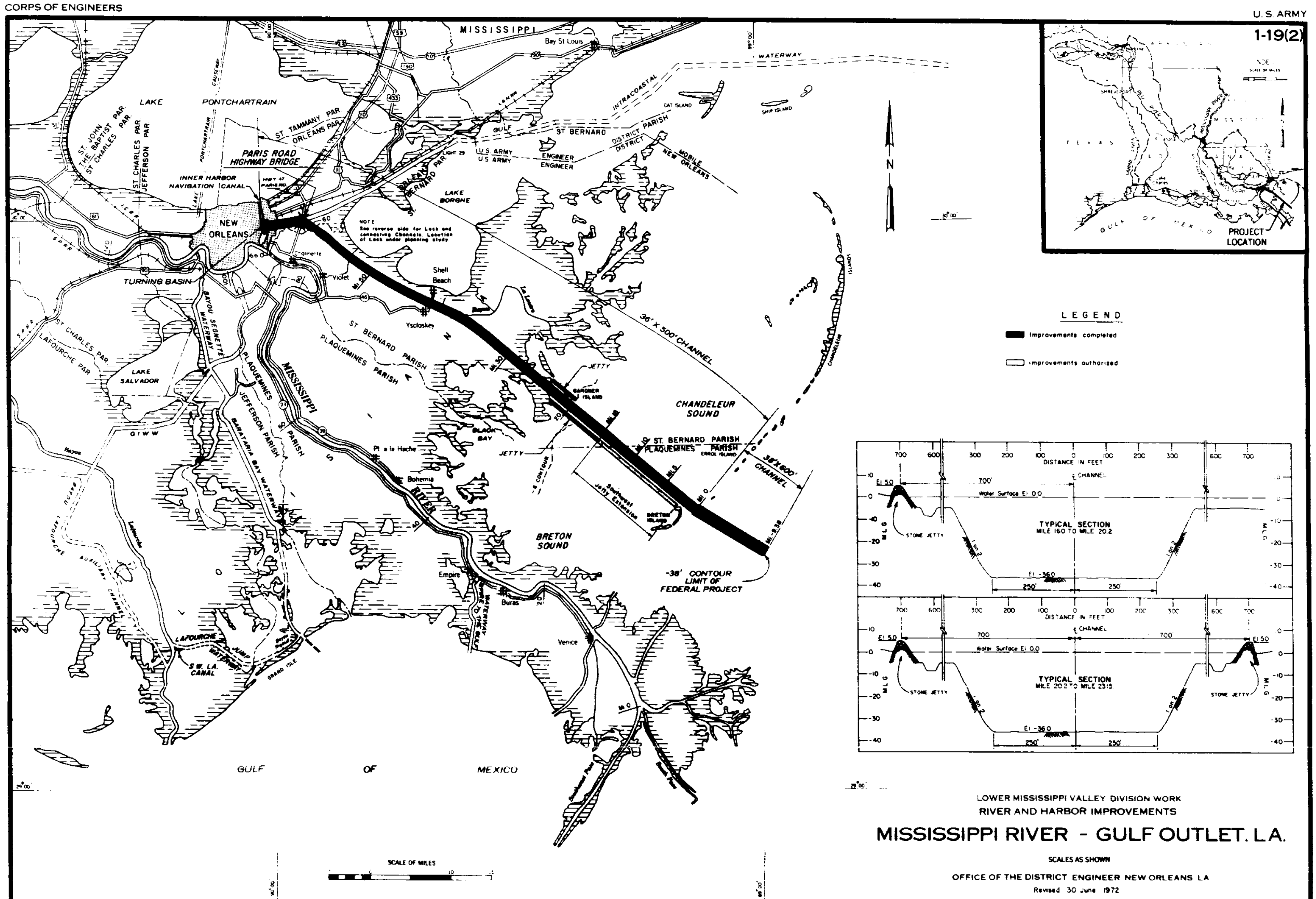

Fig. 13. Mississippi River–Gulf Outlet.

CALCASIEU RIVER AND PASS

Calcasieu River and Pass has a length of 35 miles from the port of Lake Charles to the coastline and 24 miles from the coastline to the 42-foot contour in the gulf (Figure 14). Dredge spoil from the land cut is deposited in diked areas on either side of the channel. This work is performed by cutterhead dredges, 24" to 30" sizes.

The 24-mile-long gulf entrance channel which comprises three tangents, somewhat in the shape of an "s", is maintained by hopper dredges. In the inner tangent, the agitation method of dredging is used because the material is very fine-grained and can be carried away by river and littoral currents. In the middle and outer tangents, where currents are slow, the dredge and haul method is used and material is dumped in open water west of the channel.

Calcasieu River and Pass dredging averages about 14 million cubic yards annually; 23% of the total for the District.

All shallow draft channels combined, require the removal of approximately 11.1 million cubic yards of material or 18% of the total district effort. With few exceptions, material removed from the land-cut portions of the shallow draft channels is placed in diked areas while that removed from the bay and bar channels is placed in open water adjacent to the channels.

Spoil areas are becoming increasingly difficult to obtain because of improvements on lands along the waterways. The total available area decreases each year. As a result, dredge spoil must be pumped longer distances. Although booster pumps have not been needed thus far, they will doubtless be necessary at some locations in the near future.

DREDGING ACTIVITIES 1968-1972

Figure 15 is a table of the dredging quantities and costs for the 5-year period 1968 through 1972. It was mentioned earlier that maintenance dredging requirements in 1973 and 1974 were above normal. This resulted from the long and unusually high stages of the Mississippi River that occurred during those years.

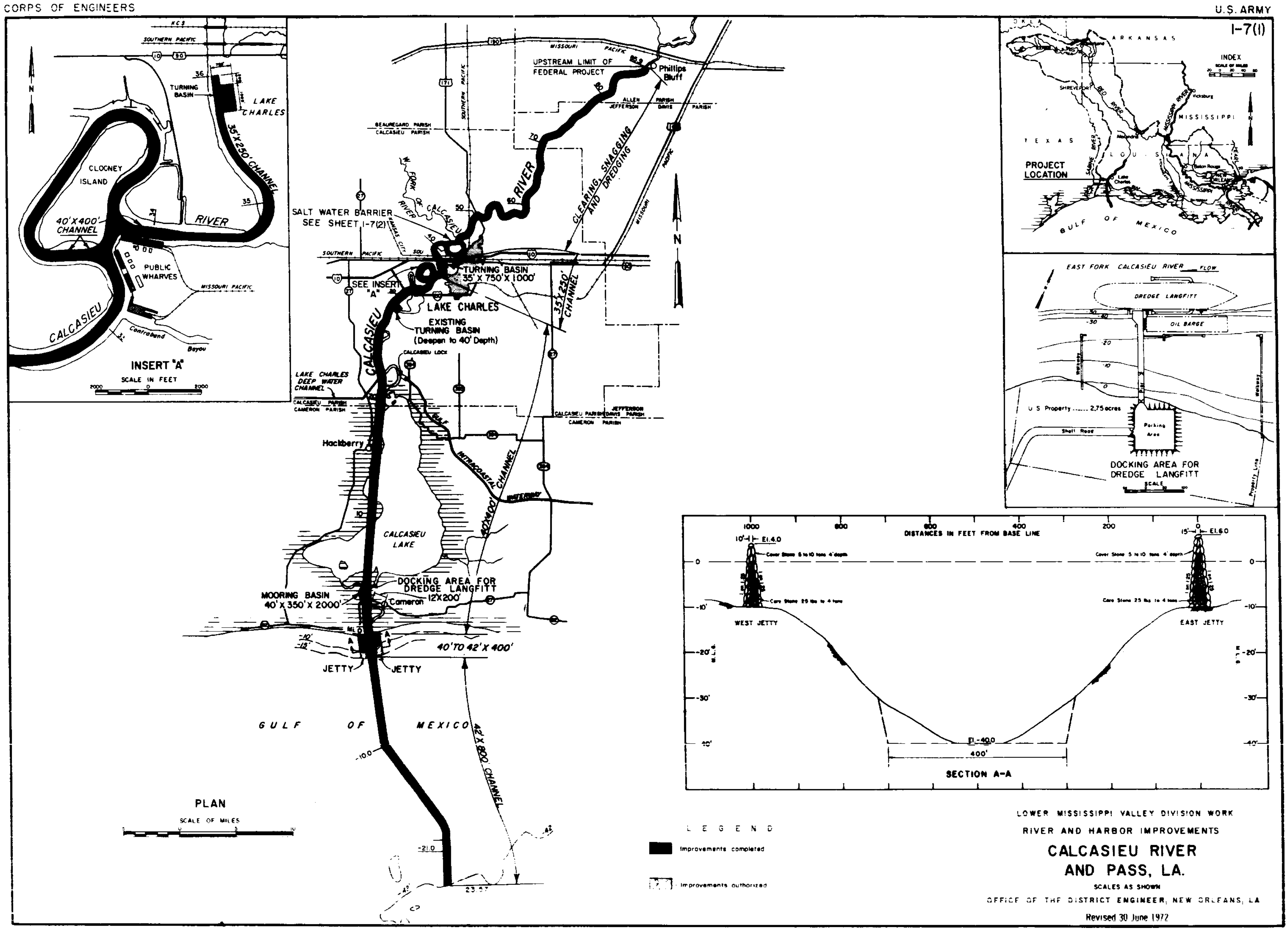

Fig. 14. Calcasieu River and Pass.

MAINTENANCE DREDGING
NEW ORLEANS DISTRICT

	1968 - 72 Average Cu. Yd. Thousands	1968 - 72 Average Dollars Thousands	Calendar Yr. 1973 Cu. Yd. Thousands	Calendar Yr. 1973 Dollars Thousands	Calendar Yr. 1974 (Partially Est.) Cu. Yd. Thousands	Calendar Yr. 1974 (Partially Est.) Dollars Thousands
Miss. River, Baton Rouge to Gulf.						
Crossings, Baton Rouge to N.O.	4,900	675	13,700	2,870	17,500	3,080
New Orleans Harbor	2,200	505	3,800	850	3,000	1,350
Southwest Pass	11,500	1,980	43,500	7,330	65,100	13,210
South Pass	2,100	310	3,500	720	----	----
	20,700	3,470	64,500	11,770	85,600	17,640
Miss. River - Gulf Outlet	15,500	2,750	5,600	1,170	9,300	2,750
Calcasieu River & Pass	14,000	1,700	4,600	1,100	5,600	890
All Others	11,100	2,350	17,900	6,230	13,700	5,700
Total	61,300	10,270	92,600	20,270	114,200	26,980
Cost per cubic yard	$0.158		$0.219		$0.236	

Fig. 15. Maintenance Dredging figures for District 1968-1972 New Orleans.

MISSISSIPPI RIVER STAGES

Figure 16 is a chart of stage-chance curves for the Mississippi River at New Orleans (Carrollton). These curves indicate the probability (or chance) of the river being at or above a particular stage for any day of the year. Superimposed on the chart are the hydrographs for 1973 and 1974. It can be seen clearly that these two years were out of the ordinary.

MISSISSIPPI FLOODS

The impact the high waters of 1973 and 1974 had on the District's maintenance dredging program is indicated by this table which compares normal dredging quantities and costs with those for 1973 and 1974 (Figure 17). The greatest increase in dredging requirements was, of course, in those parts of the Mississippi River that usually require dredging, i.e., the crossings, New Orleans Harbor, and the passes. Smoke Bend Crossing required dredging for the first time in 1973; and it was dredged again in 1974. In the reach from New Orleans to Head of Passes, where dredging had never before been necessary, there was one small area that required maintenance. That was the area from mile 2.0 to mile 3.0 AHP which is in the vicinity of Pilottown. A small hopper dredge was used for this work and spoil was deposited in Pass a Loutre.

The other areas affected by the floods were those in and adjacent to the Atchafalaya River boundaries. The greatest effect was felt in the Atchafalaya River where it flows through Atchafalaya Bay and through the bar into the gulf. Elsewhere, the Atchafalaya River caused above normal amounts of sedimentation in adjacent areas of the Gulf Intracoastal Waterway and in the East and West Access Channels in the Atchafalaya Basin. Getting back to the Mississippi River, the area presenting the greatest problem was Southwest Pass. In 1973 and 1974, dredging began earlier and ended later than in normal years. Cutterhead dredging, which usually begins in June and ends in October, began in April of 1973 and continued without interruption until the end of September 1974. During much of this time three dredges were utilized instead of the usual two.

Figure 18 compares normal dredging periods in Southwest Pass with those of 1973 and 1974. Note that the hopper dredges work earlier in the year than do the cutterhead dredges. In normal years, during periods

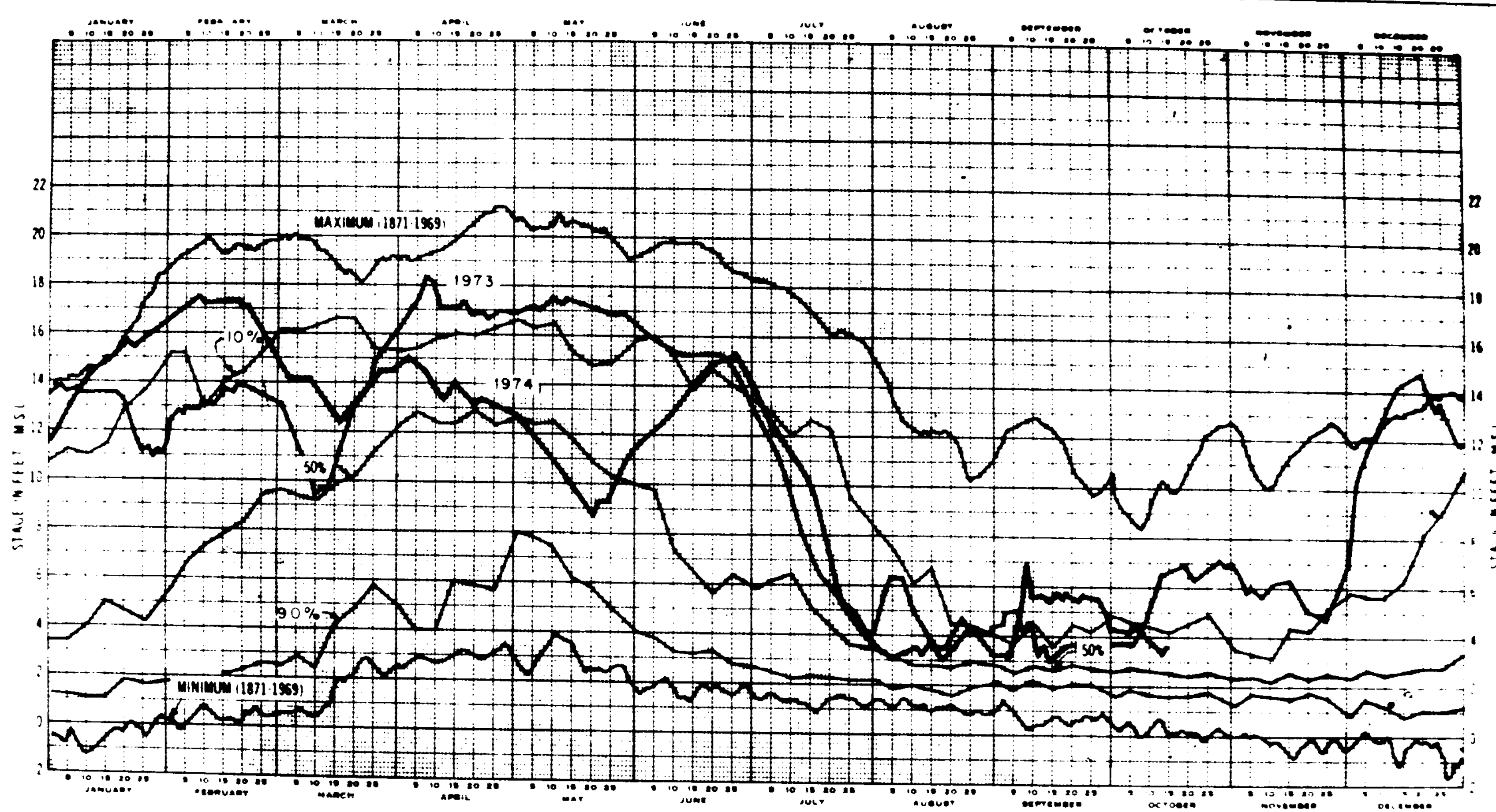

Fig. 16. Stage-Chance Curves for the Mississippi River at New Orleans.

MAINTENANCE DREDGING
New Orleans District
1968 - 1972 Average

	cu. yd. x 1000	$ x 1000
Miss. River, Baton Rouge to Gulf		
Crossings B.R. to N.O.	4,900	675
New Orleans Harbor	2,200	505
Southwest Pass	11,500	1,980
South Pass	2,100	310
	20,700	3,470
Miss. River - Gulf Outlet	15,500	2,750
Calcasieu R. and Pass	14,000	1,700
All others	11,100	2,350
Total	61,300	10,270

Average cost per cubic yard $0.158

Fig. 17. Comparison of dredging figures for 1973 and 1974 with normal figure.

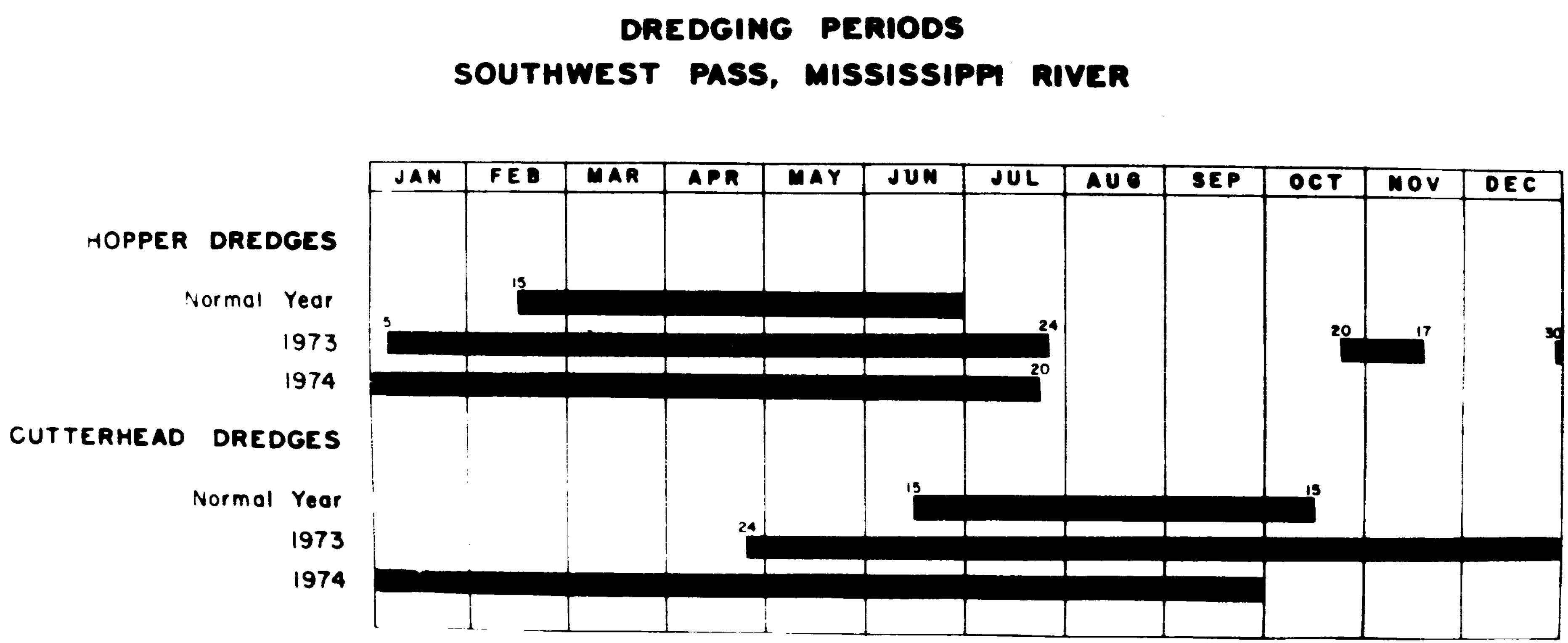

Fig. 18. Comparison of normal dredging periods with those of 1973 and 1974.

of high freshwater discharge, extensive shoaling occurs in the bar and jetty channels where the hopper dredges work. As freshwater discharge decreases, shoaling occurs farther upstream where the cutterhead dredges work. The location of the saltwater wedge comes into play here. In 1973 and 1974, the shoaling pattern changed considerably. Shoaling began early over most of the length of Southwest Pass. There was a period in the spring of 1974 when 4 hopper dredges and 3 cutterhead dredges were working simultaneously within the 20-mile length of Southwest Pass. A fifth hopper dredge worked between mile 2 and 3 AHP in the main stem of the river. It was probably the most intensive maintenance dredging effort in the history of the Corps of Engineers. The five hopper dredges constituted 1/3 of the Corps' entire hopper dredge fleet.

These two unusual years brought with them some unusual dredging practices. In the Head of Passes area, the characteristics of the currents during the high water periods were such that it would have been unsafe for a cutterhead dredge to work in the east half of the channel (if indeed the dredge could be held on station) without closing Southwest Pass to traffic. To cope with this situation, a hopper dredge was assigned to the east half of the channel while a cutterhead dredge worked the west half.

Figure 19 shows the cutterhead dredge PAUL F. JAHNCKE and the hopper dredge LANGFITT working together as described above.

In the jetty and bar channels, where it had previously been thought that only one hopper dredge could work safely, it was found in 1973 that two hopper dredges could work together safely and effectively. And in 1974, to the surprise of some, it was found that three hopper dredges could safely work in the jetty and bar channels simultaneously. One of the three dredges was the ESSAYONS, the Corps' largest, over 500 feet long. Before ESSAYONS arrived, there was some uncertainty about whether it could effectively work, even by itself, in the 600-foot wide jetty channel. Credit for the success of these complex hopper dredge operations must go to the officers and crews of the dredges.

Figure 20 is a view looking upstream in the jetty channel of Southwest Pass. In the foreground the dredge ESSAYONS is making a turn. A short distance upstream and to the left is the hopper dredge GERIG;

Fig. 19. Cutterhead dredge PAUL F. JAHNCKE and Hopper Dredge LANGFITT.

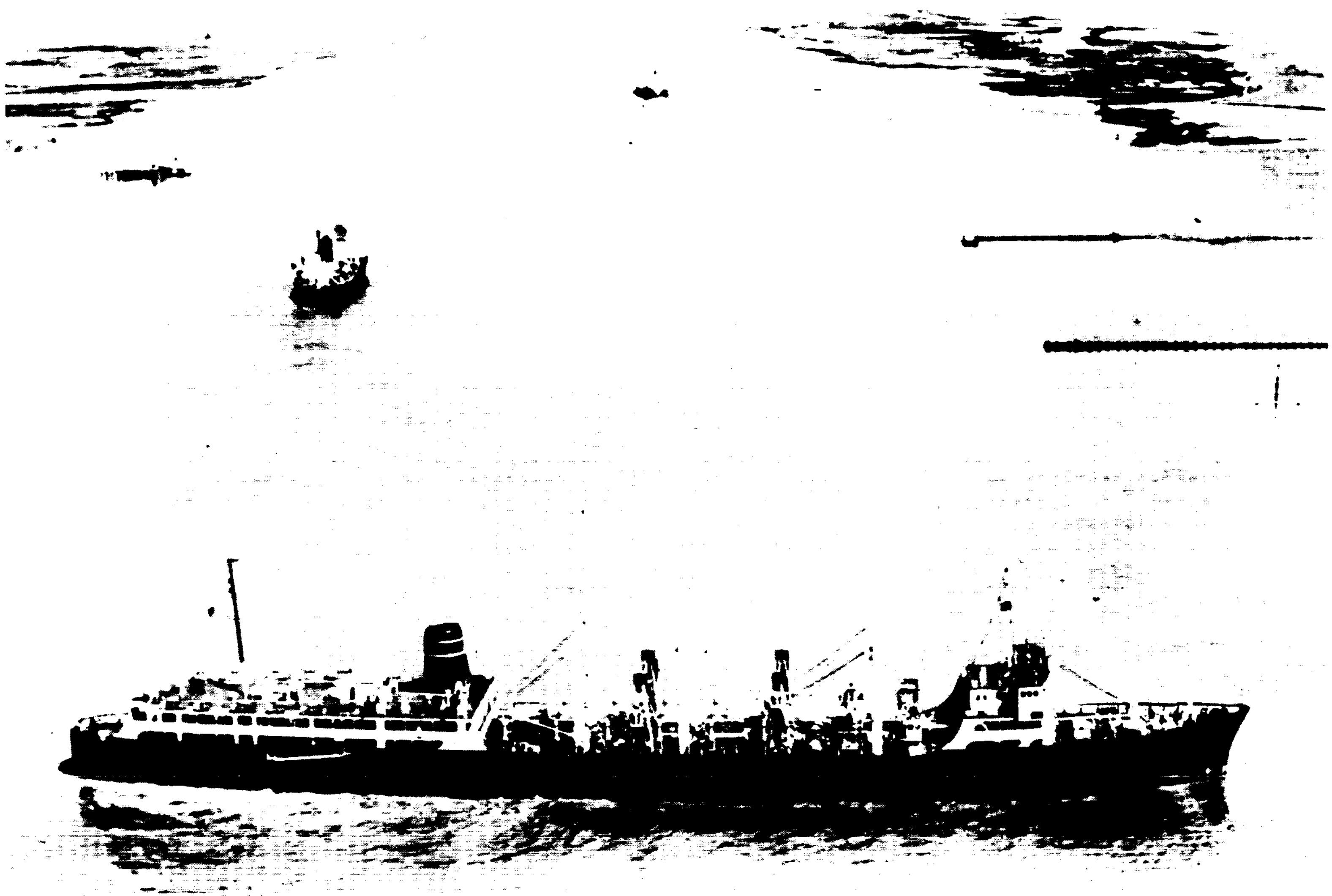

Fig. 20. Hopper dredges ESSAYONS and GERIG.

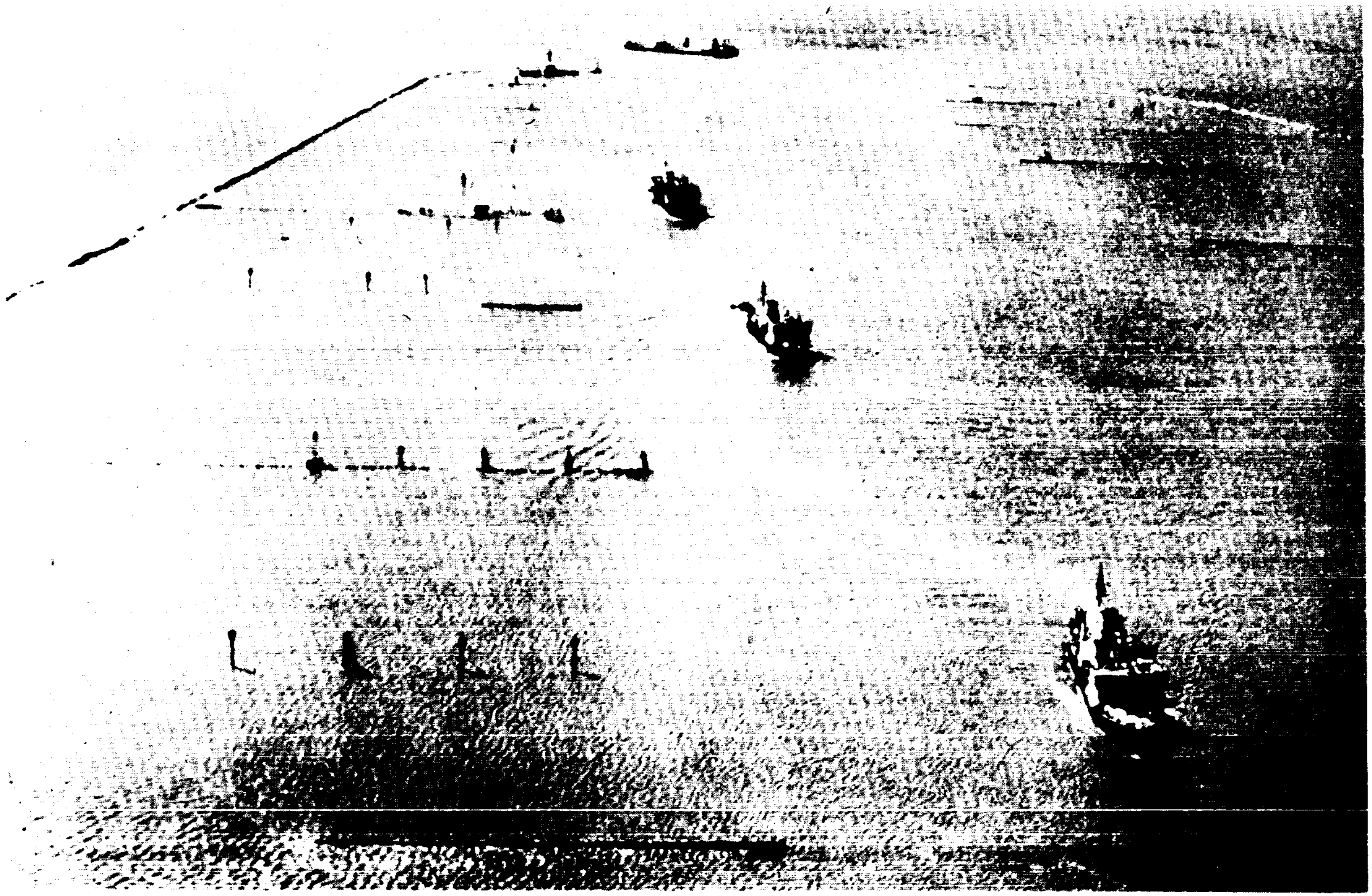

Fig. 21. Looking downstream hopper dredges ESSAYONS, LANGFITT and GERIG; also the grounded tanker MARYLOU.

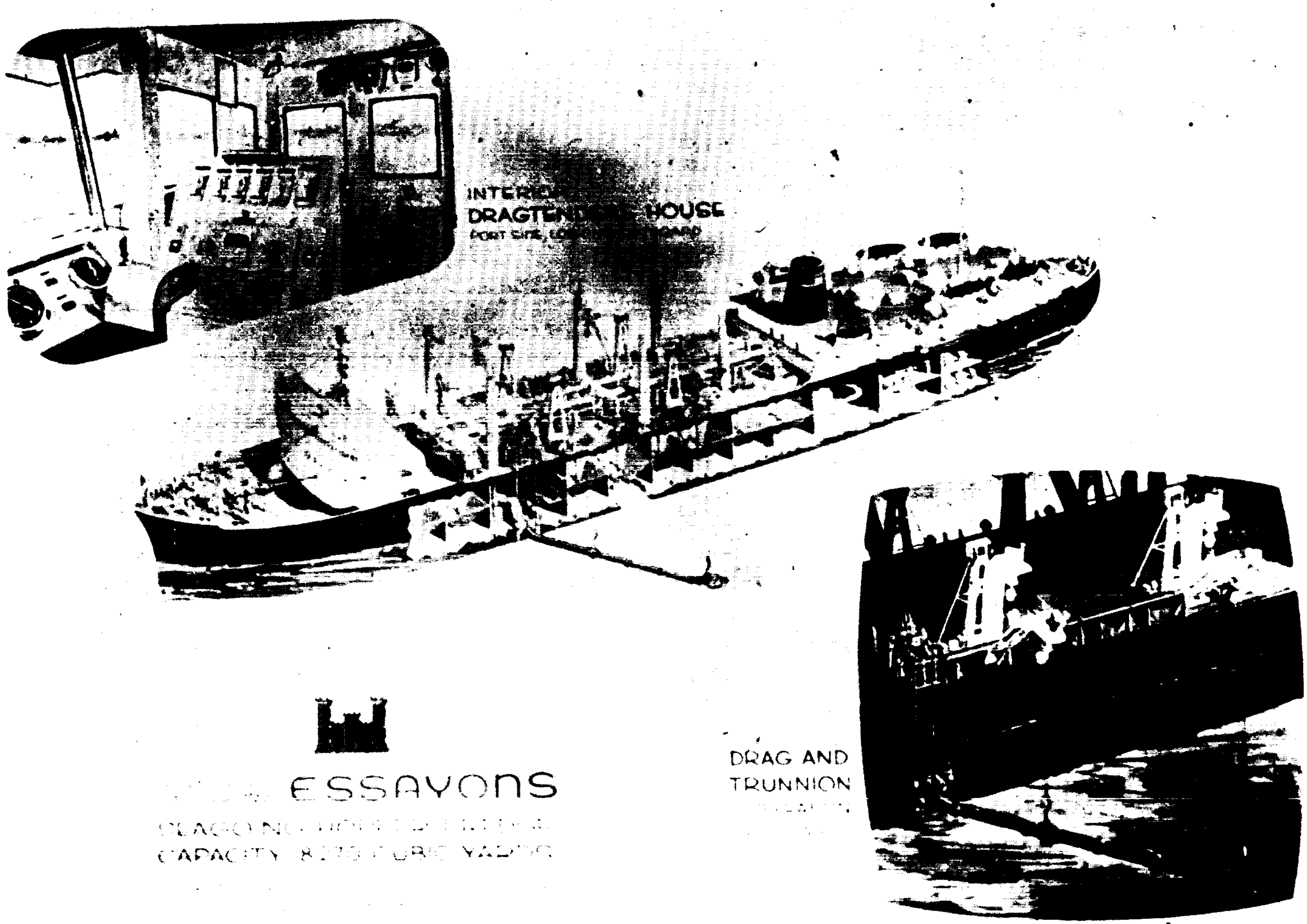

Fig. 22. Drawing of the hopper dredge ESSAYONS.

further upstream is an unidentified upbound ship.

In Figure 21 we are looking downstream over the lower jetty channel of Southwest Pass. The hopper dredges ESSAYONS, LANGFITT and GERIG are upbound on a dredging run. In the background the tanker MARY LOU is aground. She appears to be in the channel but is actually mostly west of the channel on the bar and only partially in th channel. Traffic was restricted during the grounding but the pass was closed to traffic for one day only when ground tackle was placed across the channel to pull the ship off.

Figure 22 is a drawing of the ESSAYONS showing the arrangement of the hoppers, the port drag arm and the interior of the drag tender's house.

A model study of South and Southwest Passes is now underway at the Waterways Experiment Station in Vicksburg, Mississippi. One of the purposes of the study is to find ways of reducing shoaling in the presently authorized channels. Another purpose of the study is to evaluate a deeper channel to the gulf.

Consideration is being given to providing a deeper channel from the Gulf to New Orleans and Baton Rouge. If this should materialize, dredging requirements will increase. The optimum channel size has not been determined. Channel depths from 45 feet to 70 feet are being considered. If a 55-foot channel is provided, it is estimated that annual maintenance will increase by 75,000,000 cubic yards. This would more than double the New Orleans District's maintenance dredging program.

USE OF CATAMARAN HULLS FOR SEA-GOING CUTTERHEAD DREDGES*

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ABSTRACT

Conventional, river cutterhead pipeline dredges are not designed for operation in open waters under wave conditions. There is a need for development of seaworthy pipeline dredges capable of operating in waves up to 6 feet in height. A dredge operating in the open sea will perform six characteristic motions caused by waves: rolling, pitching, yawing, surging, swaying and heaving. Stresses caused by waves on a ladder and on the connection between the discharge pipe and the floating pipeline are of particular importance.

A suggestion is made to increase the stability of a cutterhead dredge by employing a catamaran twin-hull design. A possibility of a truss design used on offshore mining projects is also mentioned.

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THE PORT OF NEW ORLEANS AND ITS DREDGING PROGRAM
TEXAS A&M SEVENTH DREDGING SEMINAR

By

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New Orleans has held the position of a world port throughout its history. Situated as it is near the mouth of the Mississippi River and being the natural outlet to the sea for the entire midcontinent, it is understandable that during the 1840's and '50's it was probably the most active port in the world. It was already more than 100 years old when the coming of the steamboat and the cotton trade started cargo flow to and from every country.

Since the Louisiana Purchase in 1803 made the lower Mississippi River the responsibility of the United States, the Federal government has invested more than $2 billion in the development and control of the river.

The Mississippi River and tributaries system includes 12,500 miles of navigable waterways and provides the drainage system for 31 states and 2 Canadian Provinces. In addition to these waterways which converge on New Orleans, the Gulf Intracoastal Waterway, some 1,100 miles in length and with 5,500 miles of tributary waterways, also feeds into New Orleans and provides a grand total of more than 19,000 miles of inland waterways that can provide access to the Port of New Orleans. In 1973 there were 10,000 ocean-going vessels and 90,000+ barges moving over these waterways through the Port of New Orleans area.

New Orleans is the Nation's second port with more than $6.7 billion annually in foreign trade, and the largest port on the Gulf Coast. It ranked as the world's third largest port based on the movement of more than 136 million tons of cargo through its area in 1973. Thirty-seven thousand people work in port services or facilities. And the port is the largest industry in the State. The total economic impact on Louisiana when cargo lands amounts to $3.6 billion a year. There are 25 miles of facilities spread over three waterways -- the Mississippi River, the Inner Harbor-Navigation (Industrial) Canal and the Mississippi River-Gulf

Fig. 1. High shot of city looking from downtown New Orleans east toward Slidell, La.

Outlet.

One entrance to the Port of New Orleans is through Southwest Pass at the mouth of the Mississippi River. The controlling depth is 40 feet. The line of river wharves for the port is 15 miles long with 35 feet or more of water alongside and 40 feet or more available in the main river channel. More than 10 miles of wharf-terminals or 88 shipping berths are publicly owned and preferentially assigned to particular steamship lines and agents. Most of the wharves are quay-type, covered, general cargo facilities.

Access to New Orleans from Lake Pontchartrain is through a 5.5 mile long Industrial Canal, with 30 feet minimum depth extending from the Mississippi River to the lake. Ships travel via the Industrial Canal Lock.

The direct access to the Inner Harbor-Navigation Canal and additional port development in the tidewater area of the port is provided by a 76-mile, 36-foot-deep by 500-foot-wide channel, called the Mississippi River-Gulf Outlet (MR-GO).

How does the Port of New Orleans stand up to the challenges of competition and new shipping techniques such as LASH and Seabee? The answer is a 30-year, $400,000,000 development plan called Centroport USA. This plan has to be realized on schedule because the growth rate of the Port exceeds the national average and it has not been possible to build fast enough recently to meet the needs of the annual increase in cargo. This is one reason why, during the past several years, the Port of New Orleans has had to act promptly and spend large sums of money on redevelopment.

The port area may be geographically compared with the great port of Rotterdam/Europort, which handles some 37,000 ships a year. The Europort idea began as recently as 1957, with construction beginning one year later. Today, 5,300 acres of land are covered almost completely with port facilities. As early as 1962, Rotterdam reached the top of the list of world ports, with 25,000 ships and 100,000,000 tons of cargo annually. Rotterdam/Europort is expected to handle 800,000,000 tons a year by the year 2000 and its vessel draft is already down to 62 feet, compared to a potential 55 feet in the Mississippi River.

Centroport USA is expected to emulate Rotterdam in the United States. Rotterdam acted fast. To reach its goal, the Port of New Orleans must also move fast.

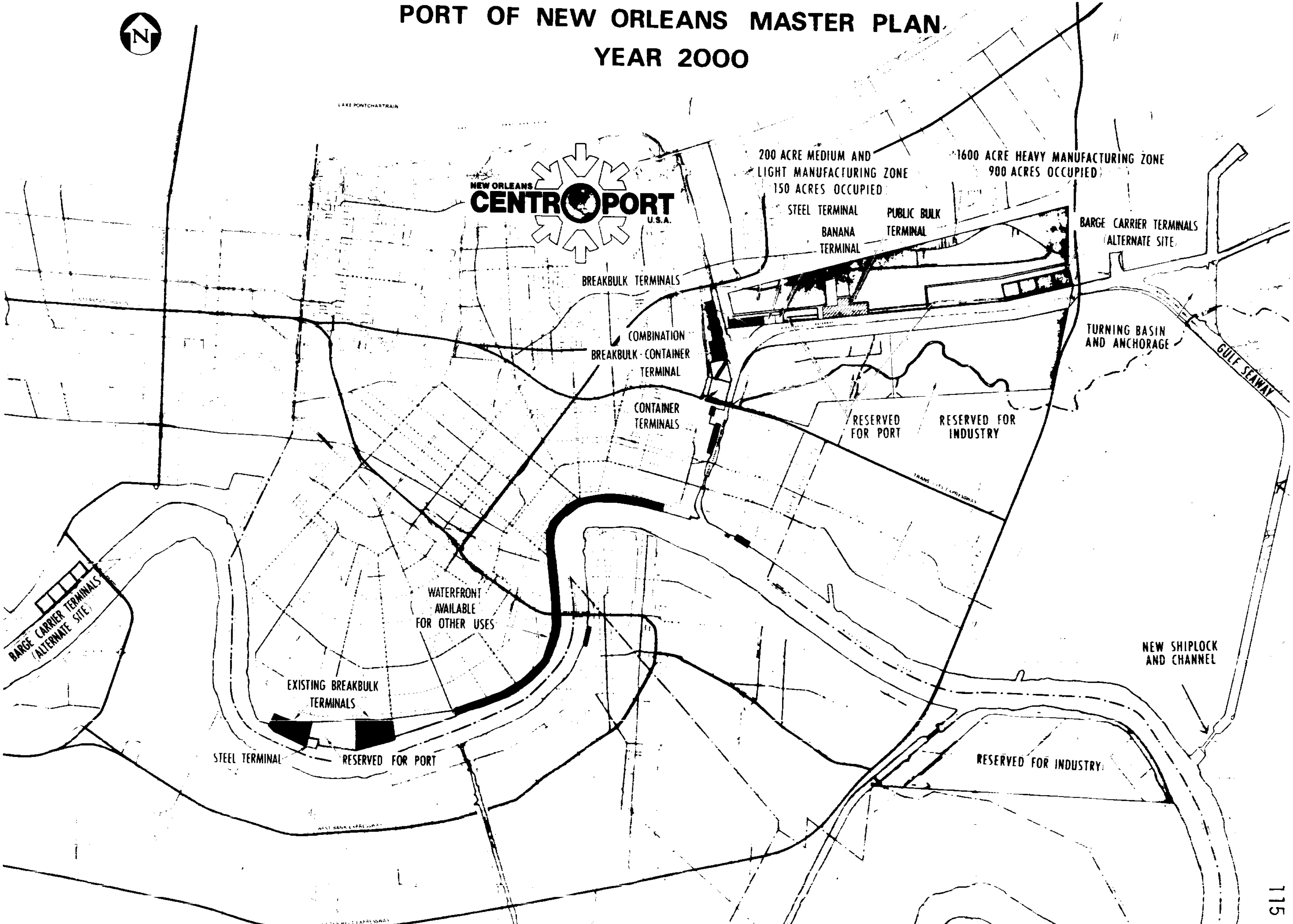

Fig. 2. CENTERPORT master plan for Port of New Orleans year 2000.

Several years ago the previously mentioned long-range development plan was launched by the port which will provide it with facilities to service the new container, LASH/Seabee, and giant cargo ships now coming into world trade. By the year 2000 the port must be able to ship, receive and distribute a three-fold increase in general cargo, All modes of transportation — ship, barge, rail, highway and air are involved. Projects of several federal, state and municipal agencies are to be combined with port projects to create Centroport USA.

The Centroport USA scheme includes a partial move from the congested bank of the Mississippi River to a tidewater area along the Mississippi River-Gulf Outlet. The second berth of the France Road Container Terminal was completed this year and the first berth, with its 30-acre marshalling area, was completed and placed under lease in 1973. The overall terminal will contain nine deep-draft berths with some 280 acres of upland development. An adjacent barge slip offers the possibility of a barge terminal in connection with the container terminal. The new barge carrier vessels will have a 20% capacity for containers, so they should also have access to a container terminal.

At the Public Bulk Terminal the rail yard and the open storage area have been enlarged and additional handling equipment has been provided as called for in the long-range plan. Further improvement of this facility will depend upon the completion of the new Mississippi River/MR-GO Lock.

The Centroport USA plan calls for barge carrier terminals in the tidewater area and on the river. The location of the initial terminals will be determined by the adequacy of the connection between the two parts of the port. The present connection through the Industrial Canal Lock has reached total capacity. In 1973 the lock handled 26,300,000 tons, passing 58,909 bottoms which required 12,616 lockages. In 1974 this tonnage may approach 30,000,000 tons and 65,000 bottoms. Projected future port commerce shows an increasing demand of 1,000,000 tons a year through this lock for the next 50 years. This increase cannot take place until a new lock is constructed. The average delay time for vessels at the lock at present is 5.6 hours. A new ship lock and connecting channel, as proposed in the long-range plan, is now being designed. Completion is scheduled for 1981.

Not only the changing technology of shipping, but also the rapidity of this change required an updating of the Centroport USA plan. The shift to containerization has been even faster than expected. The "standard size" container continues to grow larger and heavier. Container ships are getting larger - a ship holding 5,000 containers is now being designed. The new LASH/Seabee ships carry containers as well. Supertankers of 250,000 dwt will be workhorses of the tanker fleet by 1980. Dry-bulk cargo ships are following the liquid tanker fleet in size. The 125,000 to 150,000 dwt dry bulker is already in use.

At the France Road Container Terminal, a container crane must be able to handle the new, heavier, larger containers and be fast enough to load or unload a container every 2 minutes. The Public Bulk Terminal must trans-ship increasingly greater quantities of bulk cargoes in a more dust-free manner. A barge carrier terminal must provide for the handling of containers as well as for the fleeting and working of a large number of barges.

The lack of an adequate connection between the two parts of the port could delay Centroport USA progress and could mean that the wharves along the river would have to rehabilitated to give much longer service. This work is increasingly more expensive as wheel loads of material-handling equipment continue to increase. Fortunately, the LASH/Seabee barge can be used as a small general cargo ship and can be efficiently worked even at the older wharves. But larger general cargo ships now being planned will require a larger lock than any ever built before in the United States. As a minimum, a lock 150 feet wide, 50 feet deep, and 1,200 feet long will be needed.

The superships of 250,000 dwt and larger, with drafts up to 100 feet and more, will require a Superport in Louisiana. Such a facility probably will be in operation off the Louisiana Coast by 1977. New port penetration vessels of up to 125,000 dwt and with beams up to 150 feet and drafts up to 55 feet will be serviced by the Port of New Orleans. With proper planning, the activities of the two ports will be complementary. The Superport will attract major energy-consuming industries to the region which, in turn, will produce or consume materials and supplies which can best be handled by existing deepwater ports.

Fig. 3. France Road Terminal with Mississippi River in background.

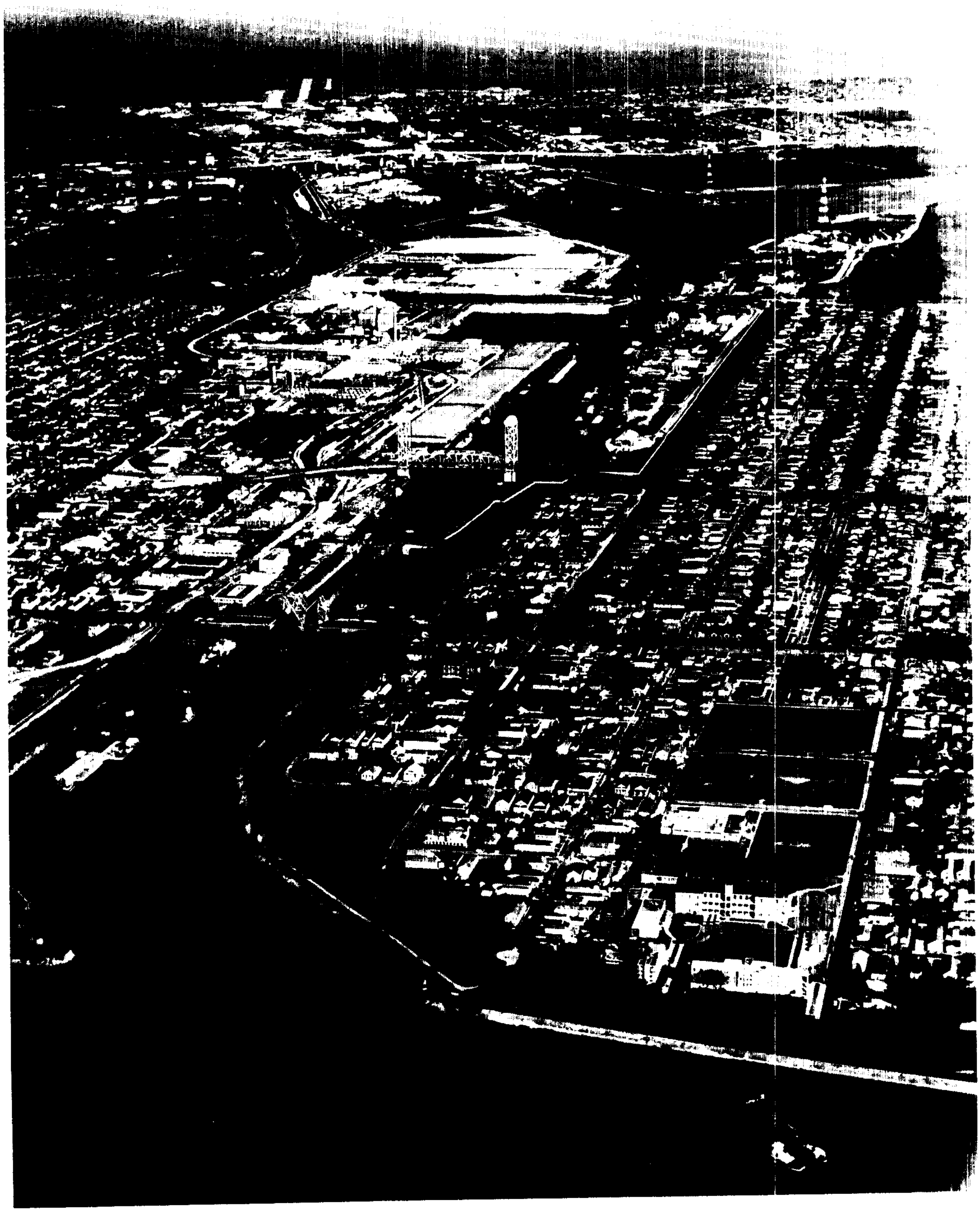

Fig. 4. Overall of Industrial Canal from Mississippi River to Lake Ponchartrain.

Dredging for the port is done in the two deep-water passes of the Mississippi River, the wharves along the Mississippi River, the Industrial Canal and the Mississippi River-Gulf Outlet. The Gulf Intracoastal Waterway (12 feet by 150 feet) also cuts through the port and a large portion of this waterway that is within the port is common with the deeper channels.

While dredging in New Orleans Harbor is normally required only 6 - 7 months of the year, the extremely high river stages of the past two years have resulted in almost year-round dredging. This dredging has been performed by contract for the past 10 years, but economics now indicate that operating our own dredge will be more advantageous. To that end, we are now in the process of acquiring a new 20-inch hydraulic cutterhead dredge. Since we dredge an average of 1,500,000 cubic yards yearly in the river and approximately 150,000 cubic yards in the Industrial Canal, we anticipate full utilization of the dredge.

Under existing laws, the Corps maintains in New Orleans Harbor, a channel 35 feet deep by 1,500 feet wide measured from a line 100-feet riverward from the face of the east bank wharves. A channel 40 feet deep by 500 feet wide is provided within the 1,500 foot wide channel. Approximately 2,000,000 cubic yards are removed annually in maintenance of the 35-foot channel (3,500,000 yards during flood of 1973). No dredging is required to maintain the narrower 40-foot-deep channel.

We have learned through years of experience that dredging in a busy port like New Orleans becomes a highly complex operation. Extremely close coordination between the Port's Docks Department and the Engineering Department is required to assure that berths are available for dredging and that dredging is performed at the proper place at the proper time. To determine this place and time, we maintain a sounding boat which conducts frequent soundings to establish a profile approximately 20 feet from, and parallel to, the face of the wharf. Wharves located on building banks are generally sounded on a daily basis while the others are checked on a less frequent basis. Dredging is then accomplished in conjunction with ship movements in those areas which show the greatest amount of accretion.

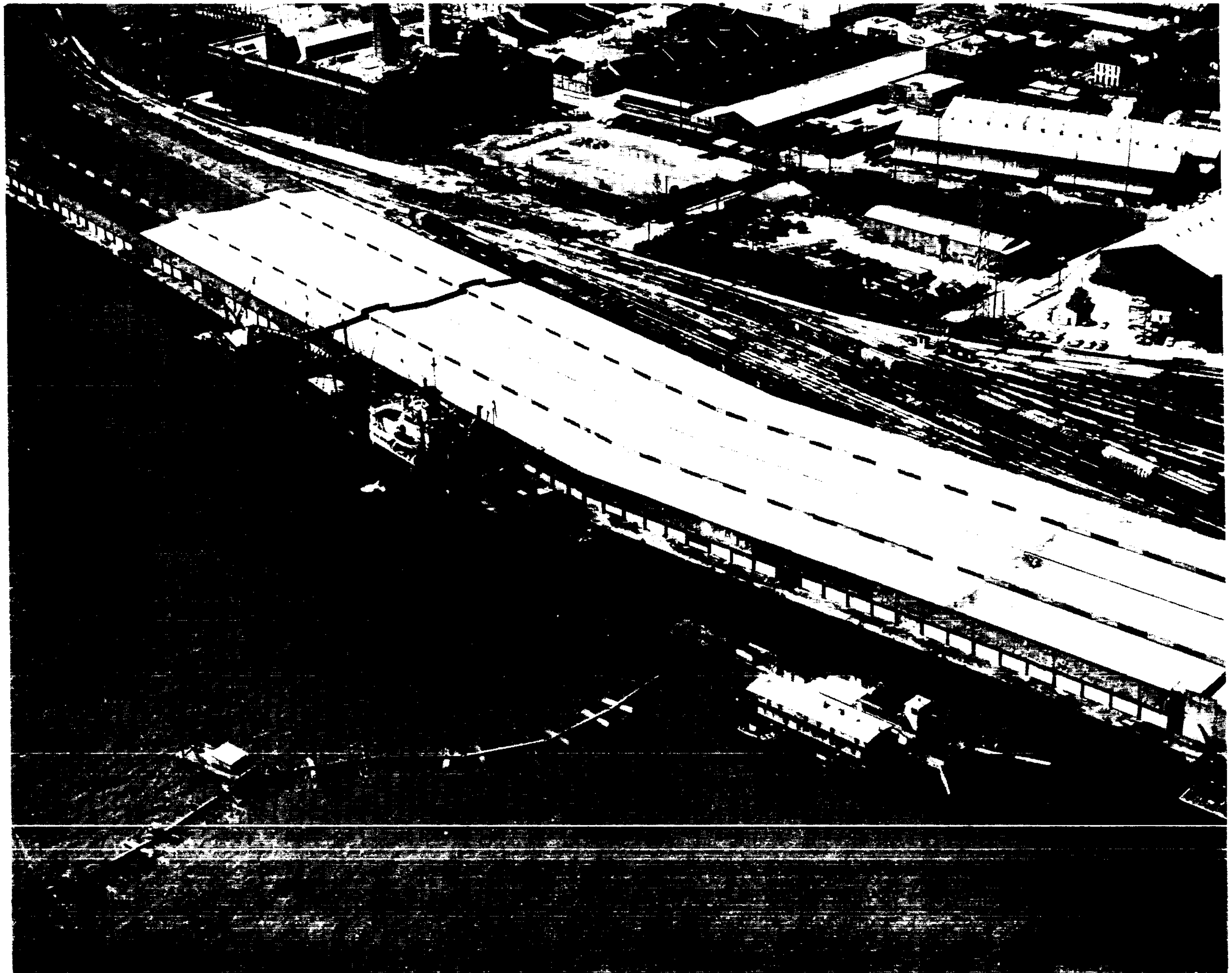

Fig. 5. Dredging operation in harbor in downtown New Orleans area.

Our dredging is accomplished using a 20-inch cutterhead dredge with a floating discharge line. Effluent is discharged beyond the 50-foot contour generally requiring approximately 800 feet of line. The material being redeposited in the river does not vary significantly from the sediment being carried in the river. In some few instances, a shoreline is required but this is minimal.

The normal procedure is to align the dredge parallel to, and 50 feet from, the wharf with the stern upstream. Thus, when the swing is complete, we have swept the 100-foot width for which we are responsible. Positioning is maintained by anchors and by lines attached to mooring bits on the wharf. Movement is downstream where possible by alternately raising and lowering spuds. Short relocations are accomplished utilizing the tenders, but longer moves require the services of a tug. On some few occasions, it is necessary to position the dredge at right angles with the cutterhead toward the wharf. This permits us to dredge a full berth when a vessel is docked at the adjacent berth.

While river sand and silt do not normally constitute a problem, we encounter considerable delay by picking up trash and debris from the river bottom. Lumber, wire rope, and especially oxygen bottles are among the better "goodies" coming to the surface. We also lose time by having to adjust spud lengths to accommodate changing bottom conditions. However, in the overall picture, these incidents are more aggravating than important.

Serious consideration has been given to other types of dredges, such as hopper dredges, suction dredges, etc. Because of the nature of our requirements, we have determined that these units will not meet our needs so we remain committed to the cutterhead dredge.

In the future, the port hopes to see a 55-foot channel provided from the Gulf of Mexico via the Mississippi River to the Ports of New Orleans and Baton Rouge, a distance of 229 miles. The Corps of Engineers has already prepared a preliminary report on this project and has found it to be feasible with a favorable benefit to cost ratio. The project, which is expected to be accomplished in the late 1970's, will involve dredging some 63 million cubic yards and is estimated to cost $110 million. The Waterways Experiment Station in Vicksburg, Mississippi, has constructed a large working model of the Passes of the River and will be conducting experiments over the next several years as to the best design features to be incorporated into the new project to hold maintenance dredging to the lowest possible levels. Additionally, the previously mentioned new $250 million lock to connect the Mississippi River and the Mississippi River-Gulf Outlet will be constructed during the late 1970s and early 1980s and will involve dredging upward to 60 million cubic yards depending on site location chosen by the Corps of Engineers.

The Port of New Orleans has a $750 million construction program planned for accomplishment by the year 2000, and the port expects to meet its goals for the future. A large part of this future will be highly dependent upon the accomplishment of major new dredging projects and an increasing major maintenance dredging program.

In conclusion it is interesting to note that if the Port of New Orleans can maintain the same rate of growth that it has in the past ten years that the value of the port's foreign trade should average $43.5 billion per year for a grand total of $2.3 trillion during the next 50 years. This is why the port's development program is so important to this Metropolitan Area, the State of Louisiana and the Nation.

PROBLEMS ASSOCIATED WITH CONSTRUCTION OF SUBMARINE PIPELINES

By

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INTRODUCTION

A few years ago a dredging contractor was installing a 10 foot diameter submarine pipeline in a lake at water depths ranging up to 120 feet. During the winter shut down, several sections of this pipe-weighing 40 tons each-were displaced by storm action. A subsequent survey showed some of the pipe sections had moved more than 50 feet. The displaced pipes were filled with silt, making them so heavy that it was necessary to clean them out before they could be lifted off the bottom. Of course, the joints and connections were damaged.

In another case, on a channel widening job, a dredging contractor cut several pipes with a cutterhead dredge, even though the pipes were supposedly placed several feet below the specified dredging depth. The pipes were installed in the bottom of a trench nearly 20 years before they were severed, but the trench was left to be backfilled by natural processes.

On yet another job, a 4-foot diameter pipeline was installed in a dredged trench in water depths ranging from 3 to 40 feet. Infiltration checks made at the end of each day and at job completion indicated tight water proof joints. Yet, only a few months after job completion a pumping check indicated that the pipeline was clogged. Investigation showed several sections of the pipe were displaced vertically and laterally; one section was found completely turned around with the bell at the original spigot end.

Many similar problems associated with submarine pipelines are found in the technical literature. The question which arises is: Did these difficulties result from contractor error or negligence, were they caused by design shortcomings, or were they simply "Acts of God" which no one could foresee? Unfortunately, the contractor usually gets rights of first refusal when it comes to blame. It is just as unfortunate that the contractor often attracts this blame by taking a few shortcuts, by failing to precisely follow specifications, or simply by not discussing potential problems with the owner and engineer before difficulties occur.

The purpose of this paper is to point out some of the submarine pipeline problems, and a few of the solutions. Most of the information presented is well-documented. Unfortunately it does not appear to be well publicized.

PIPELINE CHARACTERISTICS

Many problems are associated with the characteristics of pipelines now being constructed, as well as the materials being utilized.

Consider for the moment a pipeline typical of those used several years ago. A small diameter steel pipeline might have a 6-inch O.D. and a wall thickness of 3/8 inches. Such a pipe weighs about 22.6 lbs. per lin. ft. In pipeline design, the weight-volume relationship would be expressed in pounds per cubic foot of displacement. For this pipe, one cubic foot of pipe volume (exterior) would weigh about 115 pounds. Thus the pipe is said to have a unit weight of 115 lbs. per cu. ft. If this is divided by the unit weight of water (62.5 lbs. per cu. ft.) the pipe specific gravity is obtained, 1.84 in this case. The entrapped fluid adds to overall pipe weight. If this fluid is water, then the water-filled pipe will have a specific gravity of 2.60.

As the demand for services increases, pipe diameters also increase. At the same time, new alloys and fabrication methods allow for decreased wall thicknesses. Thus, the above pipe might now be replaced with a 12-inch diameter pipe for increased capacity, but with the wall thickness remaining at 3/8-inch. This pipe will have a specific gravity of 0.95 when empty and 1.83 when filled with water. Thus, the pipe becomes more buoyant, although it is still not capable of floating in water.

Now consider a larger pipe, such as might be used to carry cooling water for a power plant or sewage effluent. Such a pipe will generally have quite thin walls since the internal pressure is very low. A 10-ft. O.D. steel pipe with wall thickness of 1/2-inch or even less is common. When water-filled the specific gravity of this pipe would be 1.11. Fiberglass-reinforced polyester pipes of 16-20' diameter are being considered for conveying cooling water at power plants. These pipes have specific gravities of 1.01 to 1.02 when water-filled; if placed in sea water, they would be buoyant.

Not only were the older pipes smaller, they also had rigid joints, often of telescoping variety, held together by several bolts, thus giving a certain amount of rigidity to the pipeline. Today, bell and spigot joints

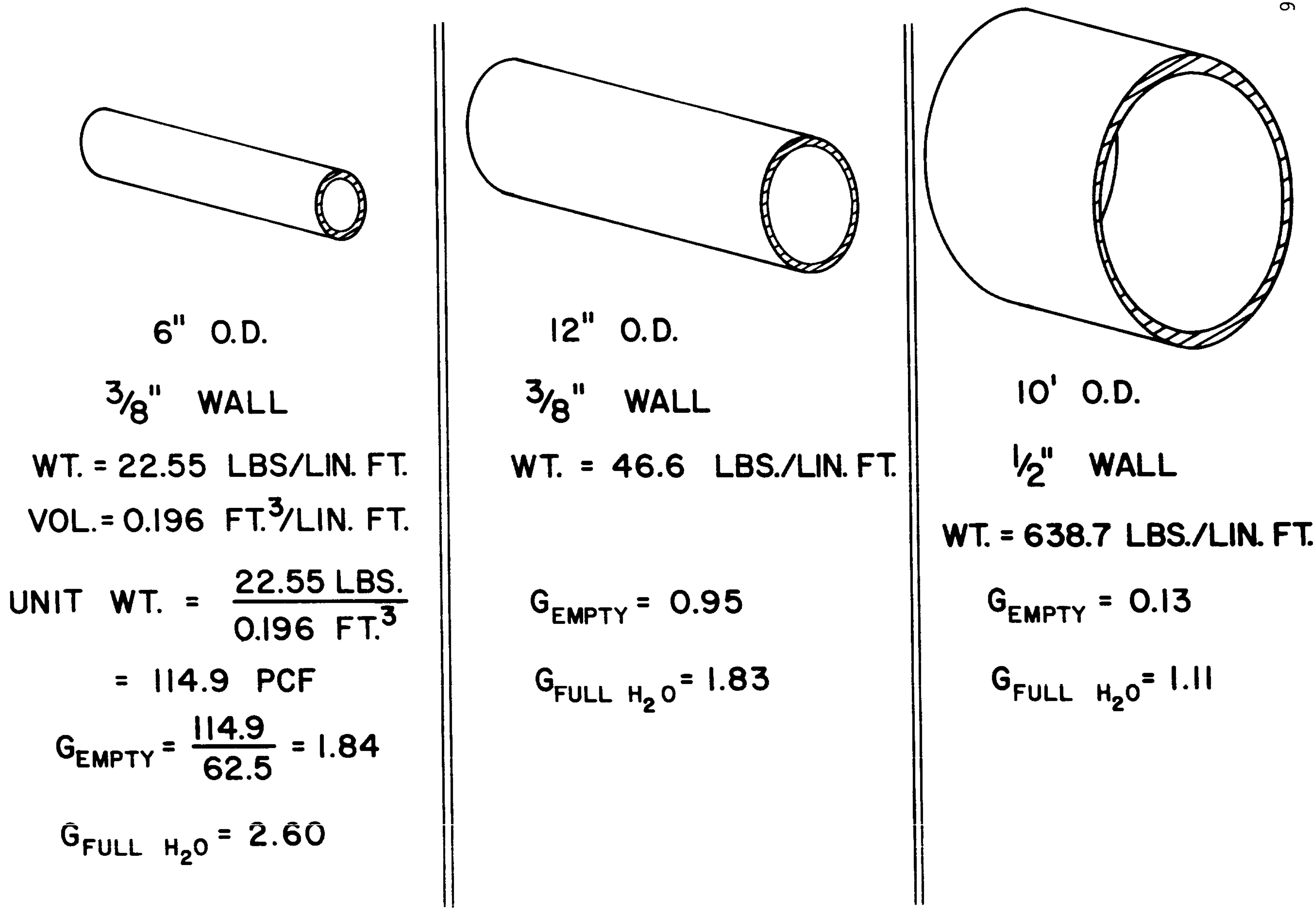

Fig. 1. Specific gravity for different pipe diameters.

with rubber gaskets are common. Bolts are used only to draw the joints up and seal the gaskets. One recent submarine pipeline design had only two bolts at the joints - one on either side at the spring line. The contractor was required to draw the joints up tight, then back the nuts off several turns. Conceptually, this allows the pipe to undergo differential settlement without overstressing the pipe. If the pipe sags due to such settlement, it will elongate, and this can be handled by the play in the joints.

To summarize, designers have developed more flexible pipes with relatively low specific gravities. These specific gravities are approaching that of water: in fact, if the pipes carry petroleum fluids, the specific gravities may be even less than one unless artificdally weighted. These events appear to at least partially set the stage for some of the problems being encountered today.

CAUSES OF PIPELINE MOVEMENT

Liquefaction

One of the more mysterious aspects of pipeline movement is attributed to liquefaction of the soil surrounding the pipe. Liquefaction has been regarded as an interesting phenomenon in the soil mechanics world for many years, and it has received significant study rather recently due to movements associated with earthquakes. In the 1964 earthquake at Niigata, Japan, a surface sand layer about 20' thick temporarily liquefied during the earthquake shock. Graves and septic tanks floated to the surface and high rise structures leaned when foundation support was temporarily lost. The 1964 earthquake in Alaska produced a liquefaction failure at Turnagain Heights which involved the movement of several million cubic yards of soil.

The studies conducted so far on liquefaction show a tremendous dependence on the state of denseness of the soil, as well as the magnitude of the shear stresses applied to the soil. The soils susceptible to spontaneous liquefaction must first exist in a metastable state of grain structure. Basically, this means that the grains are in an open network position as shown in Figure 2. When a shearing force is applied to the soil, the grains tend to roll into a denser position. Of course, water must be expelled from the soil when this occurs. If the soil is sheared suddenly--at least suddenly with respect to the speed with which water is expelled from the voids--the water pressure in the pores builds up until it equals or nearly equals

Fig. 2. Loose and dense soil particle configuration.

the applied stress. At this point, the grains are no longer carrying the load and the soil structure collapses, whereupon the soil acts briefly as a liquid with a high density of, say, 90-125 pcf or a specific gravity of 1.5-2.0. This liquefied soil is basically quicksand, although the cause is somewhat different. It is not difficult to imagine the low specific gravity pipes discussed earlier floating in this liquefied soil. Of course, when the pipe floats to the surface, it is subjected to all the other natural forces which can cause pipe damage.

Logical questions which arise are: What types of soils are capable of liquefying, how do they get in the proper metastable condition, and what causes them to liquefy? On the basis of present knowledge, it appears that the soil types capable of liquefying are cohesionless soils--that is, those which do not have some force of attraction between grains. However, they must be of relatively fine size so that the water cannot immediately escape from the voids under dynamic conditions. This limits the soils to fine sands (perhaps medium sands) and silts. Clays and gravels appear immune. However, the actual grain-size limits of susceptible soils--both on the coarse and fine side--have not been determined. Whether there are any gradation limitations such as uniformity of grain size, etc., is unknown .

The conditions leading to a metastable grain structure are not very well documented. Naturally deposited soils which have liquefied have nearly all been water deposited. The conditions of currents or turbulence which prevent the grains from rolling into dense positions probably vary with the grain size of the sediments. Fine silt grains do exhibit a small attractive force between them which acts as a glue so that one grain falling on top of another in relatively quiescent water remains there resulting in an open structural network. At any rate, normal backfilling practices in the dredging industry, such as dumping from hopper barges or releasing from a clam bucket into water, seem to produce grain structures which are loose enough to be classed as metastable. In truth, it seems to be the dumping or backfilling process that sets the stage for the ensuing liquefaction.

The state of stress on the soil necessary to cause the structural collapse is presently under investigation at many research institutions. The stress from earthquake shocks has been known to cause liquefaction as mentioned earlier, but pipeline problems have occurred in the absence of earthquakes. Almost certainly, the stress obtains from bottom pressures due to storm waves. Beneath the crest of a wave, the hydrostatic pressure increases, and it decreases beneath the trough. In deep water, these pressures are leveled out between the crest and trough, and the hydrostatic pressure at the mudline does not vary. In shallow water, however, the bottom pressure is cyclical, varying from a maximum to a minimum as the wave crest and then the trough passes over a particular point. This induces cyclic shear stresses in the soil, but at a much slower frequency than produced by earthquakes. When a cyclic shear stress is applied to an undrained soil, the pore pressure increases with load cycle (see Figure 3) until liquefaction occurs. The slow frequency of loading produced by waves will allow some drainage, and it is difficult to anticipate how the pore pressures will build up with time under conditions of partial drainage. Model studies on two fine sands in a wave tank at Texas A&M University have shown no build up in pore pressure, but this is probably the result of the small scale of the model. We have, however, seen evidence of a lag of pore pressures compared to the passage of the wave, and on a larger scale, it is possible that this can create the necessary pore pressure conditions.

Liquefaction appears to be limited to a particular depth of water with respect to wave length. It has been rather amazing to find pipe disturbance limited to relatively short lengths with no disturbance to adjoining pipe sections. This is where the flexible joints seem to enter the picture. With a more rigid pipe, it is conceivable that bridging over the disturbed area would prevent breakup.

Preventative measures to be considered in design include burial of the pipe to a depth such that liquefaction will not occur below the spring line of the pipe, the use of anchors to hold the pipe in place (although these will not be successful if the soil below the pipe liquefies) and the use of select backfill which will not liquefy. The latter is obviously very expensive and may not be feasible. However, surface protection by way of cover stone over the soil in the liquefaction prone areas is less expensive. If cover stone is used, it is necessary that the width of the protection be adequate, probably several times the pipe diameter. Also it is important that the cover be separated from the protected soil by intermediate layers. The rules presently used for filter design should be followed.

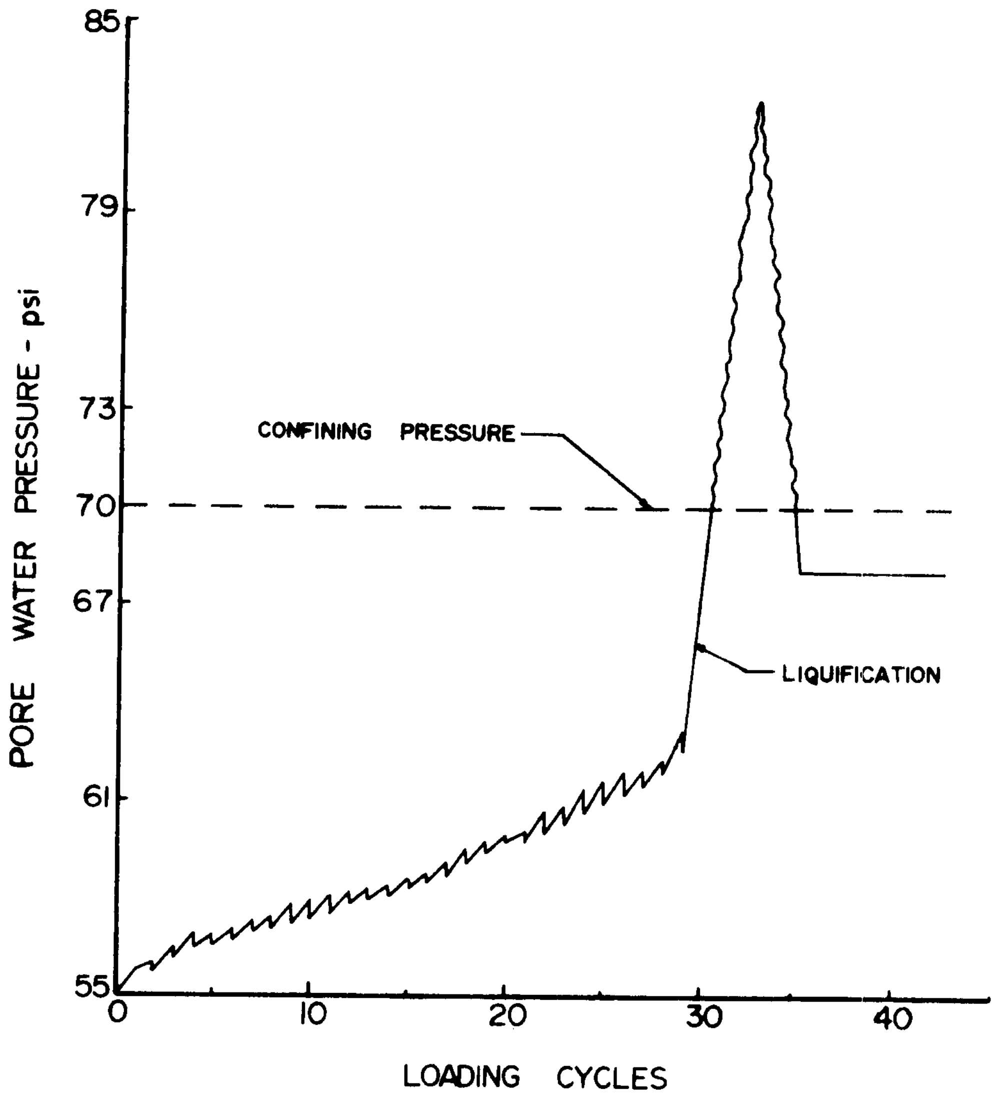

Fig. 3. Loading cycle versus pore water pressure.

Other Wave-Induced Bottom Motions

There are other bottom motions which can be caused by storm waves and which have resulted in pipe movements. The areas where these submarine movements occur on a large scale are those where the bottom sediments are very weak. The area off the mouth of the Mississippi River is one such location where these movements have occurred. When the clay carried by the Mississippi River hits the salt water of the Gulf the clays are flocculated and settle to the bottom in a group of loosely tied together particles. As much as 6 inches to 1 foot of material may be deposited each year. As a result of this rapid deposition, the clays do not have time to consolidate under the weight of overlying material and they remain in a weak, underconsolidated condition to thicknesses of over 100 feet. These clays are in a metastable condition, but the attractive forces between grains prevent a wholesale collapse as is needed to cause liquefaction.

However, the large storm waves do produce bottom forces which result in a couple or moment as shown in Figure 4. This results in a subbottom failure which can be crudely represented by a circular arc. It probably is more accurate to show a zone of movement extending to some depth below the surface since the wave will create continuous shear failures as it moves inshore.

Obviously, such subbottom movements would tend to displace pipelines. Since these movements are deep-seated, the only sure preventative method is avoiding the potential failure areas. Knowledgeable designers in the Mississippi Delta will detour around potential failure areas, which they often locate by subbottom profiling or side scan sonar.

The magnitude of these movements can be illustrated by the fact that three large offshore drilling platforms were damaged or destroyed in water depths of 270' - 290' during Hurricane Camille in 1969. After the storm, one of these platforms was found on its side at the bottom, and the other two were severely damaged, and had to be abandoned.

Scour

Much has been written about scour and it would appear that scour is a problem easily handled in submarine pipeline design. However, this is not always the case. The types of materials which are subject to scour have been well identified and charts have been developed for this purpose which

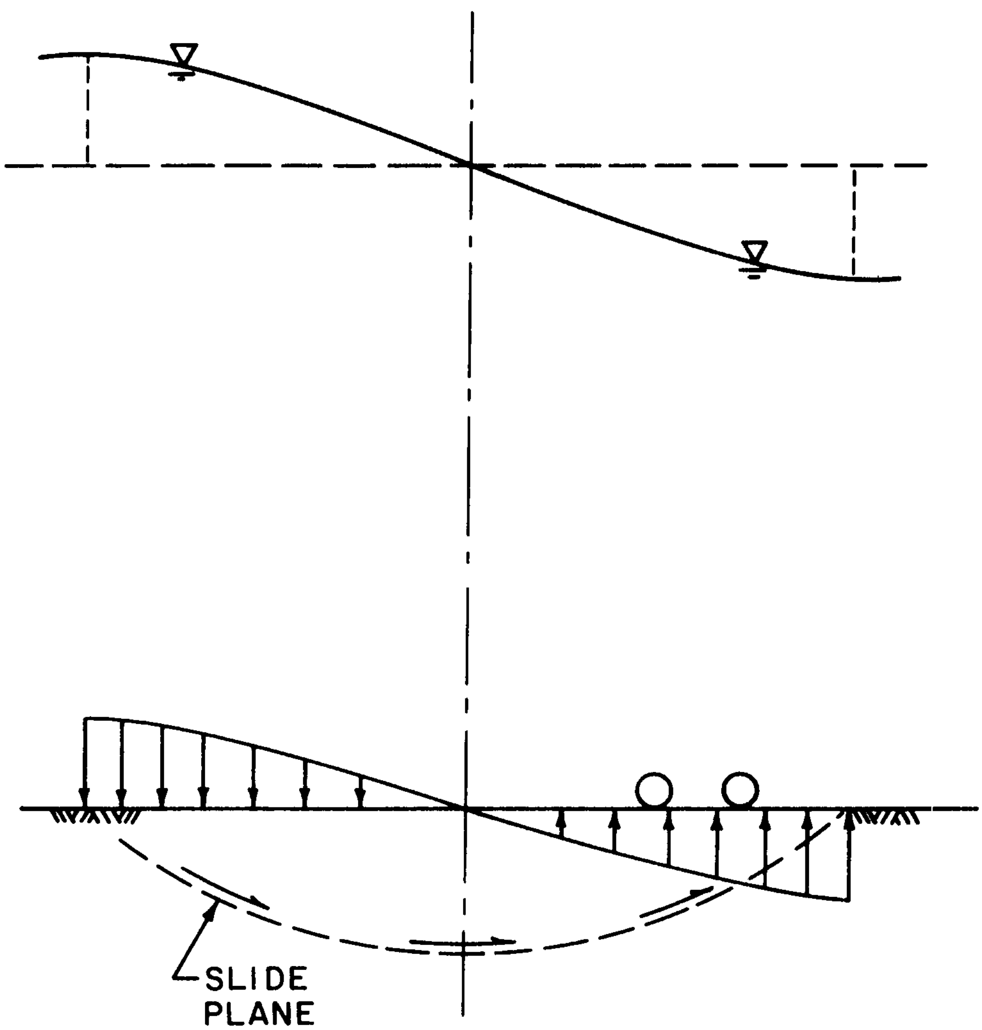

Fig. 4. Bottom forces due to storm waves.

relate current velocity to the size of particle which can be moved. The depth to which scour occurs, and the effect of foreign objects on scour patterns still needs much research. In the past, many bridge piers failed as a result of scour. Research showed scouring occurred to depths of 3-4 times the maximum height of river rise during flood. Such rules do not exist for pipelines.

The materials which are most subject to scour are the same ones most likely to liquefy, and perhaps the two proceed together in many cases. Scour in the surf zone will certainly act to remove cover around a pipe. In some cases, designers have failed to recognize the significant difference between summer and winter profiles in a surf zone (Figure 5), and pipes are exposed after periods of intense current activity. The solution here is burial of the pipeline below scour depth or covering the pipeline with a scour-proof rock or cobble blanket. Again, the blanket will not by itself prevent scour around the pipe unless intervening filters layers are used.

An associated problem occurs when pipes are placed in open trenches to be backfilled by natural processes. Currents transport sediment along the bottom until the trench is reached. At this point, the currents momentarily decrease and the bottom-moving material is deposited in the trench. Currents also push the pipe downstream slightly and some of the deposited material rolls under the pipe. When the currents decrease and the pipe returns to its original position, it is a little higher than before (Figure 6). In laboratory test flumes, we have shown that this "jacking" process can continue until the original trench is completely filled and the pipe rests on the surface at the original bottom elevation. The ASCE Committee on Pipeline Flotation makes reference to just such a case where a pipe with a specific gravity of 1.39 was placed in a 5-foot-deep trench in a river crossing and the trench allowed to backfill by natural processes. A year later the pipe was found 5 feet above its original position and completely exposed on the bottom.

<u>Bottom Slides</u>

Pipes placed on the bottom in open trenches are also subject to movement by slides of the trench wall (Figure 7). This can occur in soils which are very stiff in comparison to the materials previously discussed. Many of the stiff clays are very notorious in this respect, particularly if they contain fissures or desiccation planes in them. These clays may have been underwater for centuries, but when they are subjected to stress relief by the excavation, they tend to open up along the fissures, water gets into the fissures, and

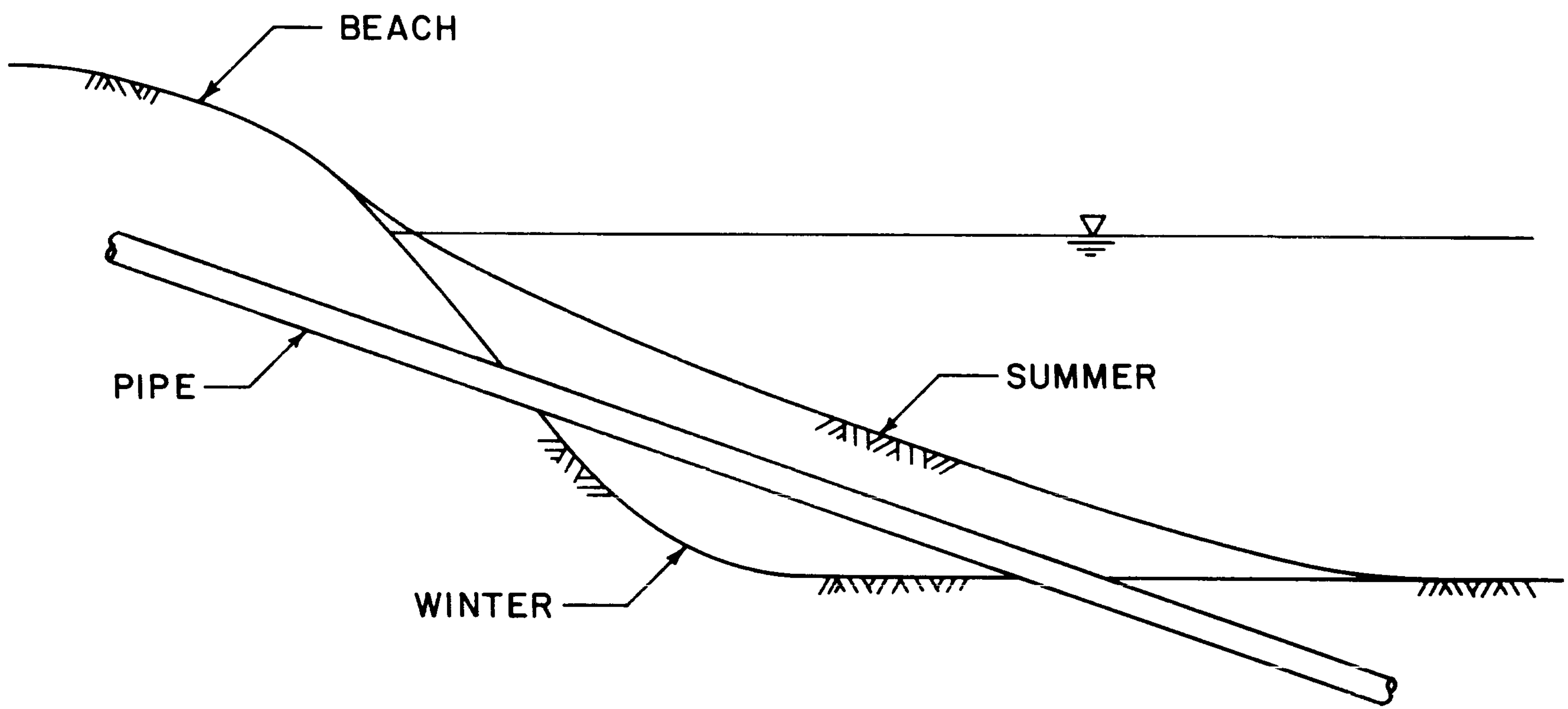

Fig. 5. Beach profile for summer and winter.

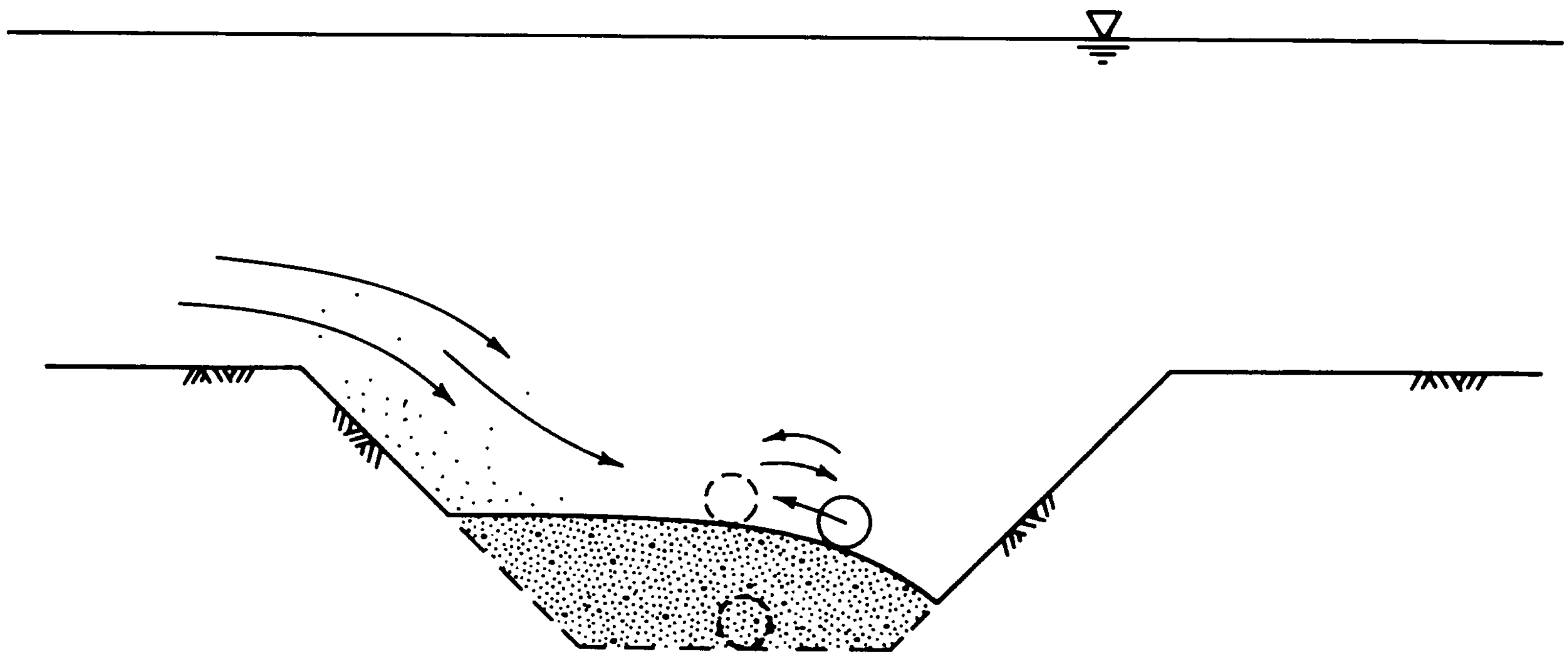

Fig. 6. Pipe "jacking" process in trench.

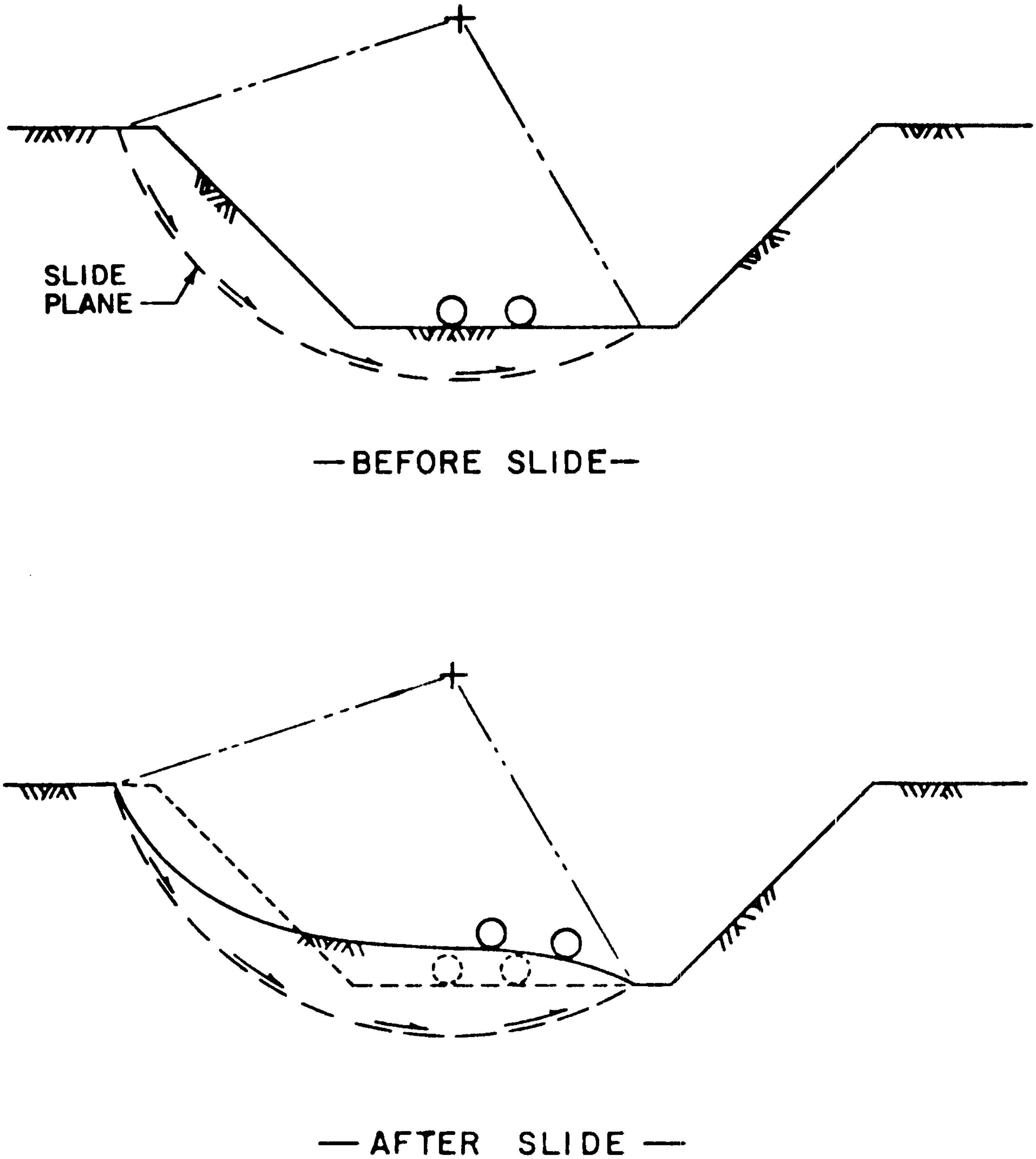

Fig. 7. Pipe movement due to slide of trench wall.

softening of the clay occurs until the trench walls slide.

Many underwater slopes are incorrectly designed because of some mistaken notions carried over from early day soil mechanics. An underwater slope will not stand at an angle any steeper than the same material on dry land, whether sand or clay. The effects of waves and currents may require an even flatter slope, as has been previously discussed.

RECOMMENDATIONS

Below are listed several points which contractors should consider when faced with construction of a submarine pipeline. While some of these are common sense, they do not seem to be considered in many cases.

1. Look closely at the soil profile presented with bid documents. Borings for design purposes should show the soil types involved. If silts and fine sands are encountered, be wary of liquefaction problems. These are soils which would be classified as SM, SP, ML and perhaps MH in the Unified Soil Classification System. The fine silty sands (SM) are probably the worst materials from the liquefaction standpoint. Consider also the effect of mixing the materials in the construction operation. Sidecasting when trenching and subsequent mixing during backfilling could produce a mixture of liquefaction-susceptible materials from layers which by themselves would not appear to be troublesome.

   On larger jobs, consider having some borings made on your own. These may show up some construction problems not foreseen by the designer.

2. Closely examine side slopes for trenches to make sure these slopes are stable during the construction period. For construction purposes, safety factors of 1.20 -1.25 are used on land. Uncertainties of wave and current forces demand higher safety factors in submarine trenches, in the author's estimation. Values of 1.30 - 1.50 seem desirable.

   When sidecasting, make sure that the material is placed far enough from the edge of the trench slope that it does not weight the crest and contribute to slope failure.

3. Backfill the trench completely and quickly. A contractor should expect that a job can be completed without undue danger of failure during construction, but lengthy periods of no backfill, or backfill up to the springline only, place the pipe in jeopardy. Do not, under any circum-

stances, leave the pipe partially backfilled during periods when storm activity is expected. Complete backfilling may not be any better than partial backfilling so far as liquefaction is concerned, but complete backfill will make a court defense much easier.

4. Remove any mounds of material or fill bottom depressions resulting from construction operations. Contractors are often desirous of leaving such bottom irregularities to be removed by natural processes and they probably will be. But if pipe movement occurs, it is difficult to prove that the features did not produce unusual currents leading to scour.
5. Perform adequate and frequent bottom surveys to show that the pipe was constructed to proper line and grade and that backfill was also properly placed to grade. This is usually the responsibility of the inspector, but the contractor may also be required to participate. Some of the newer high resolution subbottom profiling systems are capable of detecting the buried pipe as well as the bottom. These provide excellent records to show specifications were followed.
6. Consider borings into the backfill material at various stages during construction -- particularly just before shutting down for winter. Undisturbed thin-wall samplers (Shelby tubes) would be best for this task; however, many freshly deposited backfills cannot be successfully sampled without severely disturbing the soil structure. In such cases, penetration samples with the standard split spoon sampler are necessary. Make sure the driller records the distance of fall under the weight of the drill stem alone if this occurs. Classification tests (grain size and Atterberg limits) should be performed on a representative number of samples. Maintain the remaining samples until the job is completed.

COMPRESSIBILITY AND STRENGTH OF COMPACTED DREDGINGS

By

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ABSTRACT

Several series of laboratory tests were performed on three samples of polluted dredgings to evaluate their compaction, compressibility, and strength characteristics. The compressibility data are placed into perspective by comparisons with similar data from three reference soils, and the compaction and strength characteristics are compared to data reported for a variety of slightly organic soils. The compressibility of the dredgings is strongly related to dry density, and the shear strength of compacted dredged material is substantially higher than that of hydraulically-placed materials of a similar nature. In addition to the associated benefits of lower settlements and higher bearing capacities for a dredging landfill, the increased density obtained by compaction offers the possibility of effectively increasing the capacity of a given disposal area by as much as 50 percent.

Each year greater amounts of polluted maintenance dredgings are being placed within diked enclosures, and current trends in the development of urban and industrial areas indicate that the use of these dredgings as landfill material is becoming increasingly important. The two major factors that govern the feasibility of using dredgings as landfill are the technical considerations and the economic aspects. Foremost among the technical considerations are the engineering properties (such as compaction characteristics, compressibility, and strength) of the dredgings, and the work reported herein is directed toward providing an evaluation of this part of the problem. The economic aspects involve materials handling, land cost, geographic location of the site, utility value of the landfill, cost of alternative disposal methods, value of environmental benefits to be realized, etc., and a meaningful analysis of this phase of the problem can be advanced only for particular conditions associated with a specific site.

BACKGROUND

Despite the large volume of dredgings that are handled annually, there is relatively little information available on their basic engineering properties. Existing evidence indicates that, due to unfavorable long-term compressibility and strength characteristics, hydraulically placed landfills composed of unstabilized dredge spoil can not be used for extensive periods of time (often many years) to support foundation loads. The problem becomes one of improving their engineering properties as quickly and economically as possible, and several studies (McNely, 1966; Greeley and Hansen, 1969; Garbe, 1974) have investigated the effectiveness of dewatering lifts of dredgings by evaporation with or without mixing. Since compressibility and strength (and the resulting usefulness of the landfill) are strongly related to density, it is advantageous to achieve as high a density as reasonably possible. Toward this end the technique described by Garbe (1974) used bulldozers or other tracked vehicles to mechanically work the dredged material to enhance evaporation and simultaneously provide some limited degree of compaction.

One alternative approach to this problem is to dewater the dredgings (perhaps in separate dewatering bins) to approximately their optimum water content and then compact them to their maximum dry density for a given compactive effort. Whether such a method is feasible from an engineering point of view depends on the compaction characteristics of the dredgings under consideration and their associated strength and compressibility, as well as the economics of the operation. Intuitively, it might be expected that the compaction characteristics, compressibility, and strength of polluted dredgings would be similar to those exhibited by slightly organic soils, which are often encountered in field problems. In this regard Holtz and Krizek (1970) report that (a) slight changes (1 or 2 percent) in the organic content of a soil may lead to large differences in its compaction and compressibiltty characteristics and (b) soils with organic contents on the order of 6 to 10 percent, as determined by the loss on ignition test, should be avoided for the use as foundation materials. Taken too literally, this would imply that many dredgings with high organic contents should not be used as foundation soils. Despite the above indicated findings, there are situations in which the use of dredgings compacted in place may be justified. For example, the process of compaction would lead to a quicker utilization and greater stability of the landfill; in addition, the higher densities achieved by compaction (relative to the densities obtained by hydraulic placement) would effectively increase the volume of dredgings which could be placed in a given site (Krizek and Giger, 1974).

EXPERIMENTAL PROGRAM

Materials Tested

Six different materials were tested -- three dredging samples, a bedding sand, Vicksburg clay, and Grundite; the latter three were included for comparison purposes. The three dredging samples came from Riverside Site in Toledo, Ohio. As shown in Figure 1, Riverside Site is a disposal area about 2100 feet long, 700 feet wide, and 9 to 12 feet deep. The dredgings were deposited hydraulically at the northeast end of the site, and the overflow weir was located at the southwest end. As a consequence of the deposition-sedimentation process, the grain-size distribution within the disposal area is a function of the distance from the discharge pipe. Accordingly, samples were taken near the surface at locations close to the overflow weir (sample E134 near BH-16), and about 500 feet from the discharge pipe (sample E146 near BH-18). The consistency indices, grain-size characteristics, and organic content of these samples are given in Table 1.

Tests Performed

The basic tests performed within this experimental program are the compaction test, the uni-axial strain test, and the unconsolidated-undrained triaxial test. Two series of compaction tests (standard Proctor and Harvard miniature) were conducted on each of the three samples of dredged material to quantify their dry density versus water content relationships, and a third series was performed to investigate the effect of compactive effort on the maximum dry density. In one particular test sequence the data were studied in greater detail to evaluate the variation of density as a function of depth in the compaction mold. Uni-axial strain tests were conducted to determine the compressibility of each material at several different densities, and results are expressed in terms of effective stresses. The strength characteristics of the dredging samples were

Table 1. Engineering Characteristics of Dredgings Tested

Location of Sample	BH-10	BH-16	BH-18
Sample Designation	E 115	E 134	E 146
Liquid Limit (%)	76	78	68
Plasticity Index (%)	41	42	32
Effective Grain Size (mm)	0.0016	0.0011	0.0017
Coefficient of Uniformity	3.1	4.1	5.3
Organic Carbon (%)	4.0	2.5	3.0
Loss on Ignition (%)	7.3	4.6	5.5

studied by use of a conventional unconsolidated-undrained triaxial test.

Test Procedures

The compaction procedures used in this study are designated as Standard Proctor (ASTM D698), Modified Proctor (ASTM D66T), and Harvard Miniature (Wilson, 1950) with 40 tamps on each of 3 layers. In the third series of tests the arbitrarily defined low, medium, and high densities are those densities achieved by use of 20 percent Standard Proctor compactive effort (2,480 ft-lb/ft^3), Standard Proctor compactive effort (12,400 ft-lb/ft^3), and Modified Proctor compactive effort (56,300 ft-lb/ft^3), respectively, to compact dredgings with a water content equal to the optimum water content from a Standard Proctor test.

The specimens for the compressibility tests were prepared in a specially designed split mold illustrated in Figure 2. After the material was compacted, the jacket of the mold was removed, and the ends of the specimen in each of the three rings was trimmed by use of a wire saw. Then, these specimens were placed into the loading apparatus, soaked for 24 hours, and loaded incrementally by doubling the applied load (beginning with 0.32 kg/cm^2) after essentially 100% of the ultimate volume change was obtained for each loading increment. Observed swelling was negligible and assumed to not influence significantly the stress-strain relationship. Conventional triaxial tests were performed on each dredged material at confining pressures of 0, 1 and 2 kg/cm^2; specimens were prepared in a Harvard Miniature mold and tested immediately after preparation.

INTERPRETATION OF RESULTS

The results obtained from the compaction, compressibility, and strength tests on dredgings are presented and interpreted in the following sections, and the observed behavior characteristics are placed in perspective by comparing them with similar results for certain reference soils and for certain slightly organic soils.

Compaction

The compaction data shown in Figure 3 for the three dredged materials exhibit a response similar to that found for many ordinary soils; however, the dry densities are generally quite low and relatively sensitive to changes in water content, especially for the series of tests conducted in the Harvard Miniature device. The results from both compaction procedures

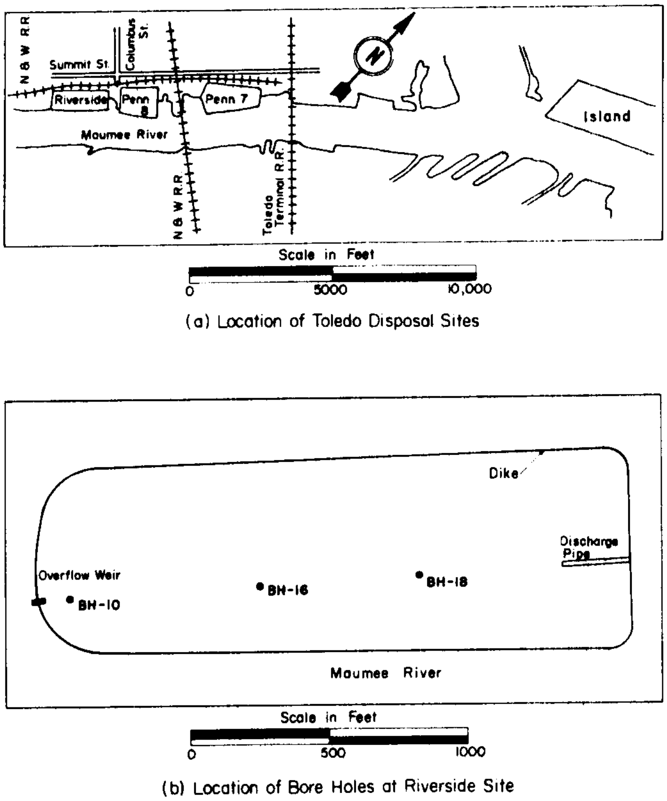

Figure 1. Location of Field Site

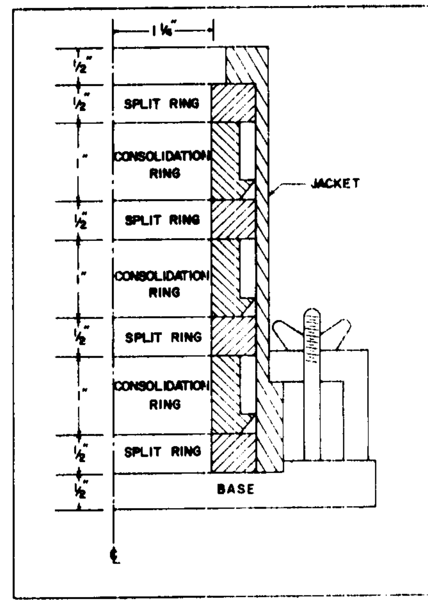

Figure 2. Cross-section of Split Ring Compaction Mold

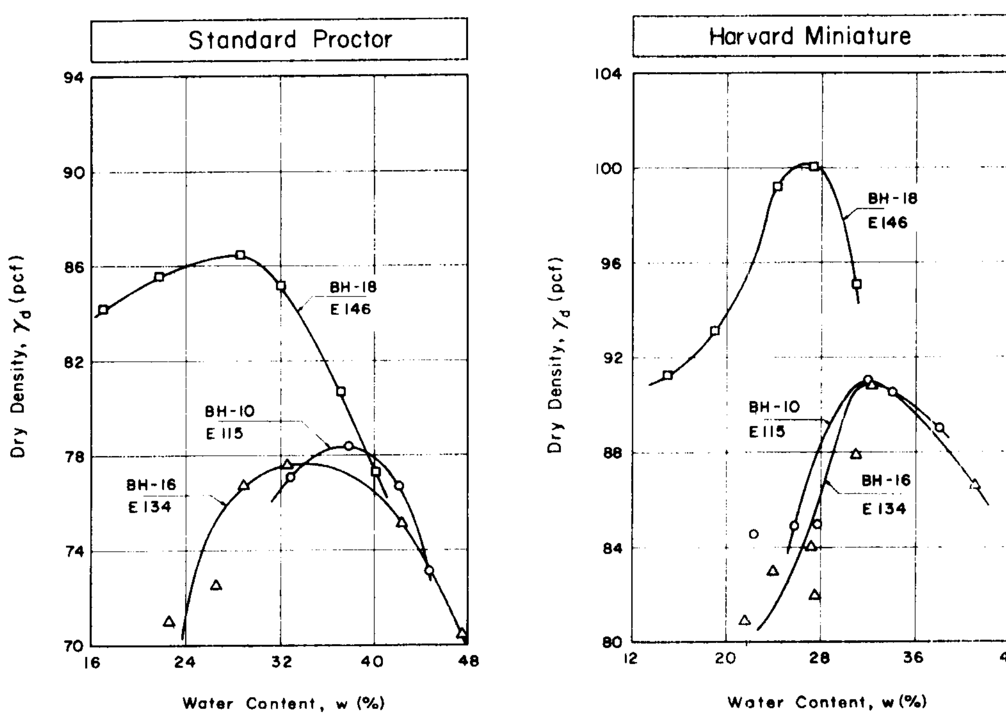

Figure 3. Compaction Curves

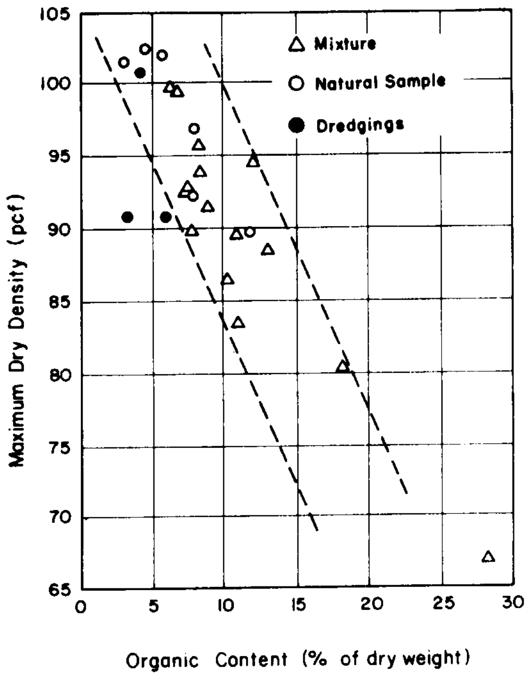

Figure 4. Maximum Dry Density versus Organic Content (after Franklin, Orozco, and Semrau, 1973)

Summary of Dry Densities (pcf)			
E_c ($\frac{ft\text{-}lb}{ft^3}$)	BH-10 (E115)	BH-16 (E134)	BH-18 (E146)
2,480	63.6	63.2	72.2
12,400	75.5	73.5	83.3
56,300	83.4	82.1	94.4

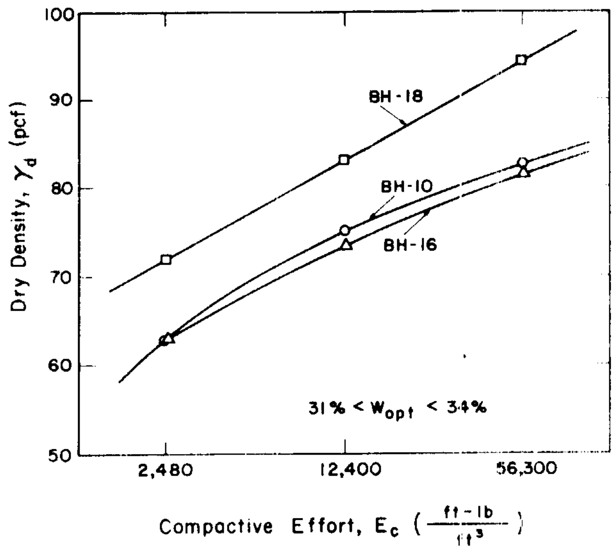

Figure 5. Dry Density versus Compactive Effort

Dry Densities of BH-16 (E134) Material (pcf)			
E_c ($\frac{ft\text{-}lb}{ft^3}$)	2,480	12,400	56,300
Top Ring	61.5	63.4	77.5
Middle Ring	63.2	73.5	82.1
Bottom Ring	64.0	70.0	86.1

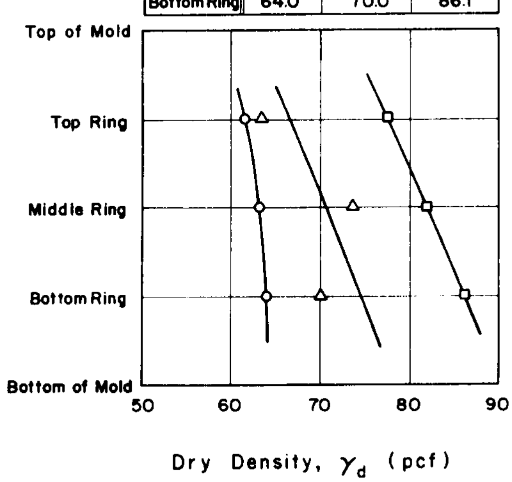

Figure 6. Variation of Dry Density with Depth in the Mold

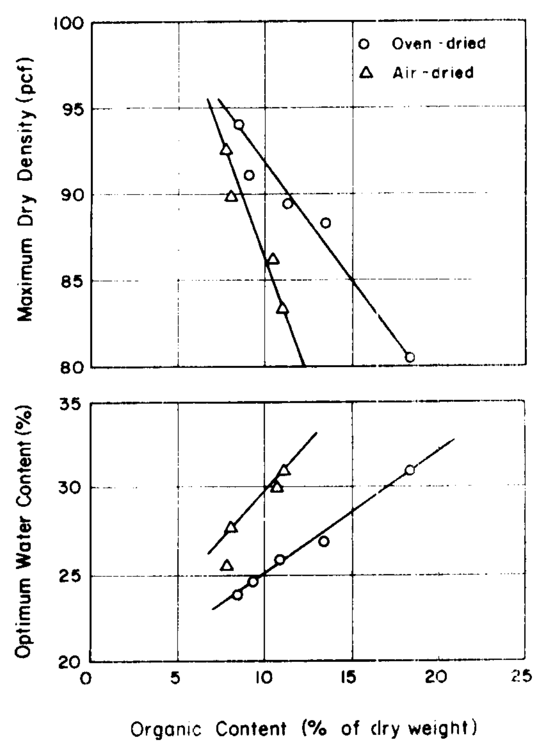

Figure 7. Comparisons Between Oven-dried and Air-dried Samples (after Franklin, Orozco, and Semrau, 1973)

indicate that the materials from BH-10 and BH-16 behave similarly, whereas the material from BH-18 exhibits a significantly higher maximum dry density and a somewhat lower optimum water content. The explanation for the higher densities of the BH-18 material is probably related to its somewhat coarser and broader grain-size distribution and its lower consistency indices.

The higher densities obtained in the Harvard Miniature test can be explained in terms of greater compactive effort and different compaction process. Although the compactive effort in a Harvard Miniature test can not be readily calculated, the normal procedure employed in this experimental program is generally regarded to impose on the soil a compactive energy greater than that developed in the Standard Proctor test. Perhaps of greater consequence is the fact that the Harvard Miniature test employs a kneading process to compact the soil, whereas the Standard Proctor test utilizes an impact type of compactive effort; the kneading process is usually considered to model more closely the actual compaction procedure used in the field. The relative response of these dredged materials can be appreciated more fully by comparing in Figure 4 the results obtained with those reported by Franklin, Orozco, and Semrau (1973) for a wide variety of natural organic soils and organic soil mixtures compacted according to Harvard Miniature procedures. As can be seen, the dredged materials have somewhat lower dry densities.

The influence of compactive effort on the resulting dry density of each of the three dredging samples is given in Figure 5 and indicates that a 22-fold increase in the compactive effort increases the dry density about 30 percent (or about 20 pcf) in each case. Furthermore, increasing the compactive effort from Standard Proctor (12,400 ft-lb/ft^3) to Modified Proctor (56,300 ft-lb/ft^3) increases the dry density about 10 percent (or about 10 pcf). Once again, the densities of the BH-18 material are seen to be significantly higher than the densities of the other two materials for comparable compactive efforts. From a broad point of view the dry density of compacted dredgings may range from 60 to 90 pcf or more, depending on the type of compaction and the compactive effort. Since the initial dry density of a hydraulically placed dredged material is usually about 50 to 55 pcf (Krizek and Giger, 1974), it becomes immediately evident that the compaction of these dredgings to a density of 70 to 80 pcf would increase the short-term capacity of a given spoil area by about 25 to 50 percent; even after 6 to 8 years in place, the dry densities of hydraulically placed dredged materials may be only 60 to 65 pcf (Krizek and Giger, 1974). The economic significance of this situation can only be evaluated for a particular set of circumstances.

In a supplemental study performed in conjunction with the compressibility tests, specimens obtained from the bottom, middle, and top ring of the split mold were used to investigate the variation of density as a function of depth. Cau and Olson (1971) studies the uniformity of laboratory-compacted samples and found considerable variations in density with depth in the mold, as well as with radial and circumferential position. As shown in Figure 6, the results of this limited study confirm their observation and indicate that the density increases with increasing depth for all compactive efforts. The compaction of an overlying layer evidently densifies the lower layers slightly through additional energy input while the lower layers are under a more confined condition. Variations in field densities are even more significant than those measured in the laboratory, and one must keep in mind that maximum dry densities determined in both the laboratory and the field represent only average values.

A final point of interest is illustrated in Figure 7, which shows that the maximum dry density and the optimum water content for a particular soil with a given organic content are affected significantly by the process used to dry the specimens. Oven-dried samples generally exhibit higher maximum dry densities and lower optimum water contents than air-dried samples. This is perhaps due to some chemical alteration of the soil constitutents as a consequence of accelerated drying, and less moisture is required to achieve a favorable arrangement of soil particles for a given compactive effort. The engineering significance of this finding is that air-dried samples should normally be used to conduct laboratory tests, because this situation more closely approximates actual field-drying conditions; alternatively, if special field procedures are employed to achieve rapid drying, the materials tested in the laboratory should be treated in a similar manner.

Compressibility

The compressibility data for the three dredging materials are presented in Figure 8, and results are seen to be strongly dependent on the density in most cases. When compared to the bedding sand, Vicksburg clay, and Grundite, the dredgings are found to exhibit a compressibility on the order of that found for a loose sand, but significantly greater than that observed for a dense sand, the Vicksburg clay, and the Grundite. In view of the relatively large silt-size fraction of the dredging samples, this comparative behavior is not particularly surprising. The general trend of increasing modulus with increasing stress or strain is a consequence of the particular testing technique, wherein the specimen is confined laterally. The overlapping curves manifested in certain cases are attributable to experimental errors, which may stem from non-uniform densities, trimming of the specimen, seating problems, etc. According to these data, a 10-foot layer of dredgings similar to those found at BH-16 compacted to a medium density (about 74 pcf in this case, using the data from the specimen in the middle ring) would settle about 2 inches under an applied load of 1000 psf. Alternatively, if the dredgings were compacted to a loose density (about 64 pcf in this case), a settlement of about 10 inches could be expected under the same load.

Strength

The unconsolidated-undrained triaxial test results shown in Figure 9 indicate that all three dredged materials exhibit essentially the same response. The stress-strain curves manifest a characteristic strain-softening pattern, and most failures were associated with a system of cracks in the specimen, as described by Gau and Olson (1971). When interpreted in terms of the Mohr-Coulomb failure criterion, cohesion and friction values are found to range from 0.5 to 0.9 kg/cm^2 and 18 to 20 degrees, respectively. However, as a consequence of the non-uniform conditions associated with the Harvard Miniature test (Gau and Olson, 1971), the above findings must be accepted with caution.

Based on the results from a number of unconfined compression tests on "undisturbed" dredging samples from essentially the same locations as BH-10, BH-16, and BH-18, cohesion values were found to be between 0 and 0.2 kg/cm^2. Hence, the limited data presented herein suggest that the

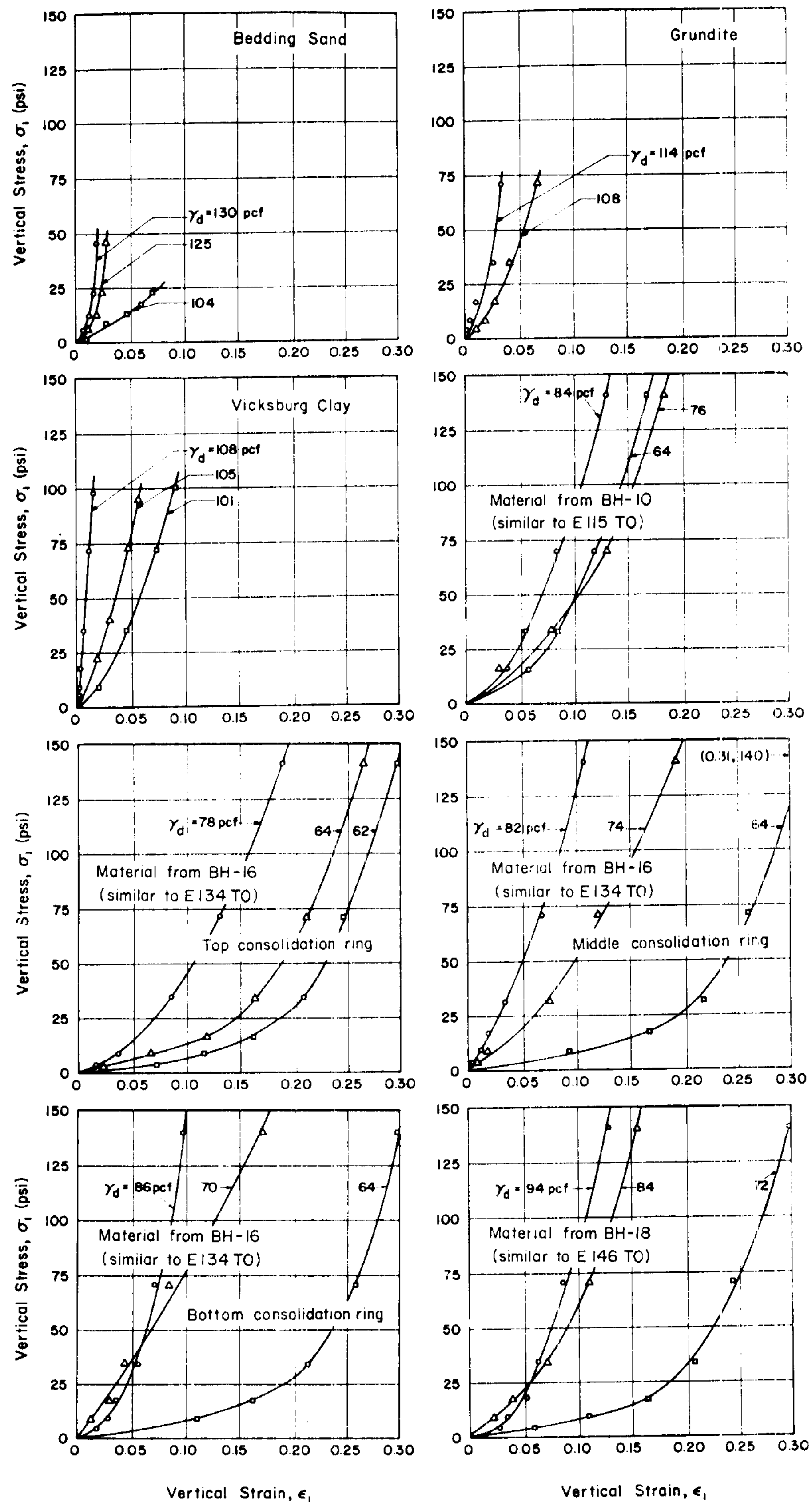

Figure 8. Compressibility of Various Soils and Dredgings

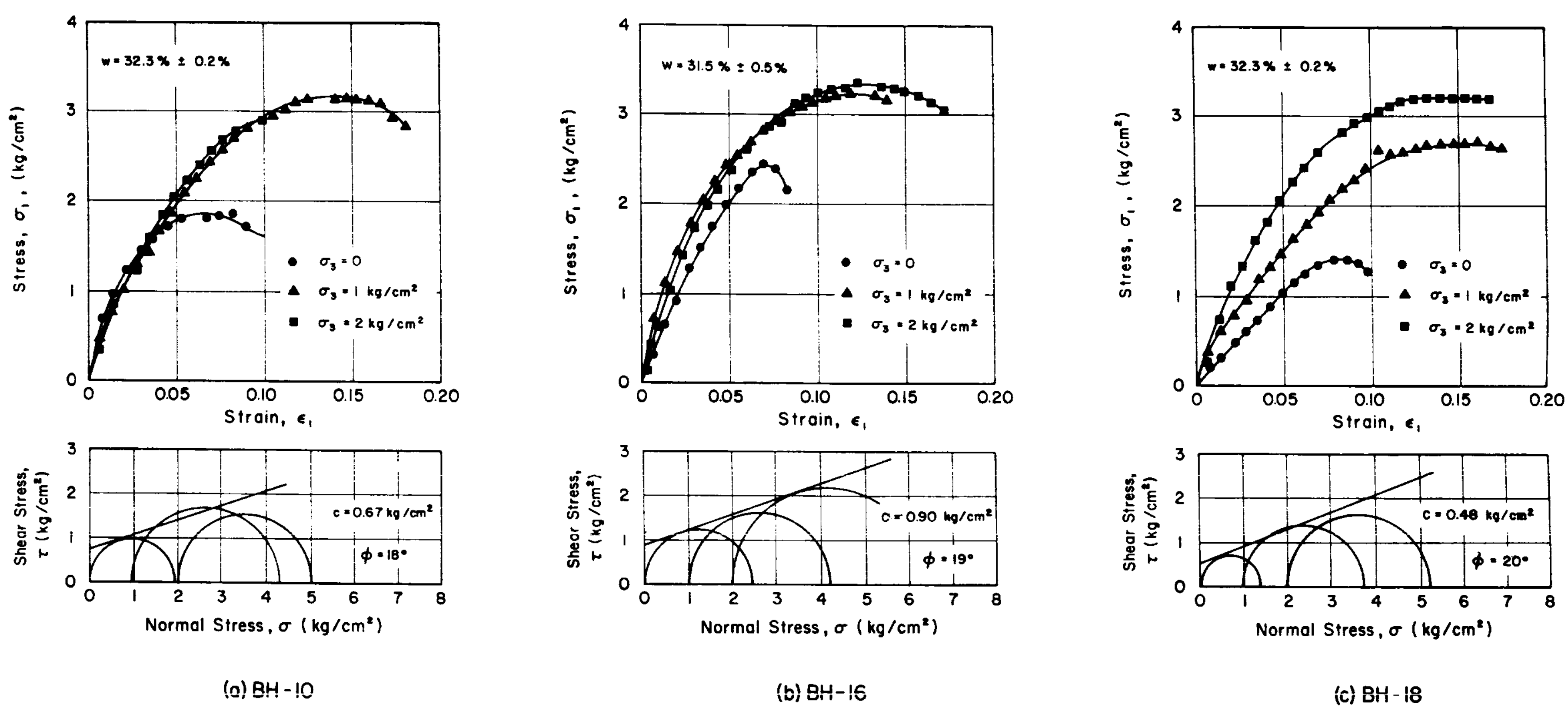

Figure 9. Stress-strain and Strength Data

compaction of dredgings will lead to a considerable improvement in their shear-strength characteristics. Although the shear strength of hydraulically placed dredgings will certainly increase as consolidation takes place, the long time periods normally required for this to occur, especially in view of the low overburden pressures, is an economic factor to be considered. Furthermore, it is doubtful that the dry densities achieved by consolidation would ever equal those of the same materials in a well-compacted condition. Although the influence of organic content on the unconfined compressive strength of dredged materials was not specifically investigated in this program, work reported by Franklin, Orozco, and Semrau (1973) indicates that the unconfined compressive strength of a material at a given dry density decreases with increasing organic content.

CONCLUSIONS

Based on a limited experimental investigation of the compaction, compressibility, and strength characteristics of three dredged materials from Riverside Side in Toledo, Ohio, the following conclusions may be advanced:

1. Upon compaction at approximately the optimum water content, the dredgings exhibited maximum dry densities of 78 to 86 pcf for Standard Proctor compactive effort, 82 to 94 pcf for Modified Proctor compactive effort, and 90 to 100 pcf for Harvard Miniature compactive effort.
2. A comparison between the maximum dry densities of compacted dredgings and the dry densities of hydraulically placed dredgings indicates that the capacity of a given disposal site may be increased substantially by compacting the materials.
3. The compressibility of dredgings is strongly related to dry density, and their load-deformation characteristics are comparable to those of loose sand and significantly larger than those of dense sand, Vicksburg clay, and Grundite.
4. Compacted dredgings exhibit shear strengths which are substantially higher than the strengths of natural, hydraulically placed materials.
5. The compressibility and strength characteristics of dredgings can be improved considerably by use of compaction procedures; the increased cost of such an undertaking relative to other acceptable alternatives must be balanced against the benefits to be realized (quicker utility, greater usefulness, environmental impact, etc) before a final decision can be made.

ACKNOWLEDGEMENTS

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PERMEABILITY AND DRAINAGE CHARACTERISTICS OF DREDGINGS

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ABSTRACT

The dewatering of disposal areas composed of polluted maintenance dredgings is directly related to the permeability and drainage characteristics of these materials, but relatively little quantitative data regarding these properties is available. This study synthesizes the results determined from several different types of laboratory tests on a variety of different dredgings, and the coefficient of permeability is found to decrease from about 10^{-4} to 10^{-9} cm/sec as the void ratio decreases from approximately 10 to 0.6; data from two field infiltration tests indicate permeability values on the order of 10^{-4} to 10^{-5} cm/sec for materials that yielded laboratory values of about 10^{-7} to 10^{-8} cm/sec. The electro-osmotic coefficient of permeability is found to be on the order of 3×10^{-5} cm^2 /volt-sec, which is about one-half the value used for a large variety of soils. A partial vacuum serves to increase substantially the dewatering rate as compared to drainage by gravity alone. The overall permeability and drainage characteristics of polluted maintenance dredgings are seen to be comparable to those for many fine-grained, organic soils.

INTRODUCTION

Current concern for the environment has imposed serious restrictions on the open water disposal of polluted maintenance dredgings, and this has led to the frequent use of diked enclosures to retain these materials. The potential usefulness of these spoil areas depends to a large extent on their ability to be dewatered, and this is related directly to the permeability and drainage characteristics of the dredgings. Accordingly, the work described herein was undertaken to enhance our understanding of these important properties and to provide quantitative data for use in subsequent evaluations. Since the bulk of maintenance dredgings are fine-grained, the major emphasis is directed toward a study of these materials.

Several series of tests have been conducted -- both in the laboratory and in the field -- on undisturbed and remolded samples of various typical dredgings from several Great Lakes harbors. After the dredgings are characterized in terms of their mineralogy, organic content, Atterberg limits, and grain-size parameters, the ensuing test series included gravity drainage, vacuum drainage, electro-osmosis, direct permeability, conventional consolidation, slurry consolidation, and field infiltration. Insofar as possible, correlations are deduced among the various dependent and independent variables, but these must be viewed within the context and limitations of the test program.

MATERIALS TESTED

The dredging samples tested in this program were taken from several harbors around the Great Lakes, but the majority of them came from the vicinity of Toledo, Ohio. In virtually all cases the materials were taken directly from fill areas or from the discharge pipes through which the materials were pumped in slurry form. Materials were tested over a broad range of water contents, depending on the stage of the disposal-sedimentation-consolidation process at which the sample was taken.

Notation

Samples are designated by a three-part symbol; the first part is a letter that indicates the general consistency of the sample (C materials have the consistency of very muddy water; D materials are thick muds; and E materials are capable of maintaining a given shape); the second part is a chronological number that denotes a sample within any given consistency group; and the third part consists of two letters that identify the city and state from which a particular sample was obtained (CO=Cleveland, Ohio; DM=Detroit, Michigan; MM=Monroe, Michigan; and TO=Toledo, Ohio).

Sampling Procedure

The disturbed samples were obtained by use of either a shovel, a flap-valve sampler, or simply a bucket; the undisturbed samples were taken by means of a manually operated, specially designed, piston sampler. Further details regarding the sampling procedure are reported by Hummel and Krizek (1974).

Classification

Since the permeability of a soil is greatly influenced by mineralogy, organic content, grain-size, gradation, etc., these characteristics are quantified for a limited number of samples which are considered to be typical of all samples tested.

Mineralogy -- X-ray analyses with heat and ethylene glycol treatments were used to determine the mineralogical composition of the clay-size particles (less than 2 microns) of seven samples, and the qualitative results are summarized in Table 1. Of the clay minerals present, the quantitative amounts of illite, kaolinite, and mixed-layer minerals were about 50 to 70%, 15 to 25%, and 20 to 35%, respectively; however, a few of the samples had only trace amounts of mixed-layer minerals.

Organic Content -- The organic content of eight samples was determined by four different methods -- total volatile solids (dried at 600° C for 1 hour after drying at 100° C), loss on ignition (dried at 440° C for 24 hours after drying 105° C), and organic carbon by the methods of Grass and Lemert (1971) and Walkley and Black (1934) -- and the results are given in Table 2. When comparing the data in Table 2, it is of importance to note that organic matter consists of about 55% organic carbon and the weight losses associated with drying at 440° C or 600° C may be partially due to losses in the mineral fraction and/or interlattice water.

Atterberg Limits -- The liquid limit and plasticity index for 19 dredging samples are shown on the Casagrande (1948) plasticity chart given in Figure 1. In general, these materials would be classified as OH according to the Unified Classification System; alternatively, a few may be classified as OL, CH, or MH, depending on their clay content and organic content.

Gradation -- The gradation curves for four typical samples (each with and without the addition of Calgon) are shown in Figure 2. Based on gradation tests from over 30 additional samples, the effective grain size, D_{10}, was approximately 0.00045 ± 0.00010 mm and the coefficient of uniformity, C_u, was about 20 ± 10 with a few values around 40. Hence,

Table 1

Qualitative Mineralogical Analyses

Sample	Minerals Positively Present	Minerals Possibly Present
C3TO	Mica (Illite) Kaolinite Vermiculite Carbonates Some organic matter	Quartz
C7MW	Mica (Illite) Kaolinite Quartz Dolomite Carbonates	Vermiculite
C8MW	Mica (Illite) Dolomite Carbonates Some organic matter	Seprolite Vermiculite
C9MW	Mica (Illite) Kaolinite Dolomite Quartz Carbonates Some organic matter	Vermiculite
C22TO	Mica (Illite) Kaolinite Vermiculite Dolomite Quartz Carbonates	
D4CO	Mica (Illite) Kaolinite Montmorillonite Dolomite Carbonates Organic matter	Vermiculite
D6TO	Mica (Illite) Kaolinite Vermiculite Quartz Dolomite Some carbonates	

Table 2

Comparison of Organic Content Determinations by Various Methods

Sample	Total Volatile Solids (%)	Loss on Ignition (24 hours) (%)	Organic Carbon (G - L) (%)	Organic Carbon (W - B) (%)
C22TO	9.03	7.60	3.72	
C27MM	12.63	11.93	7.55	8.8
D3TO	6.96	6.40	3.21	5.1
D4CO	10.53	10.00	8.21	8.8
D8TO	9.33	8.36	4.51	
D9TO	8.81	8.71	3.90	
D10TO	8.91	7.93	3.69	
E63TO	8.56	6.20		

Table 3

Coefficient of Permeability
from Short-Term Drainage Tests

Sample	Coefficient of Permeability (cm/sec)	
	Gravity Only	With Vacuum
D1DM	3.4×10^{-6}	4.1×10^{-7}
D3TO	2.6×10^{-6}	3.5×10^{-7}
D4CO	5.1×10^{-6}	4.9×10^{-7}

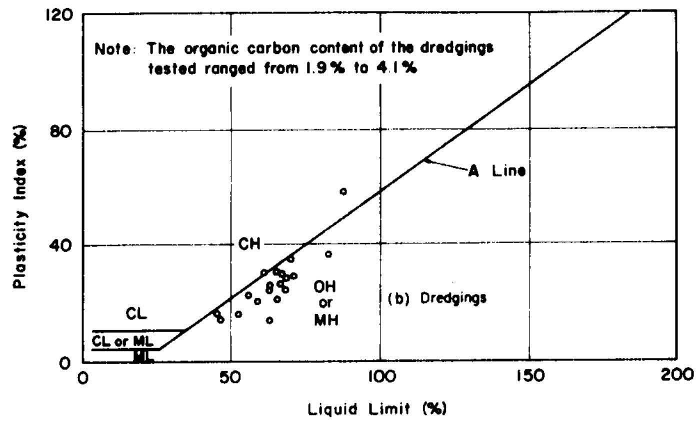

Figure 1. Classification of Dredgings on Plasticity Chart

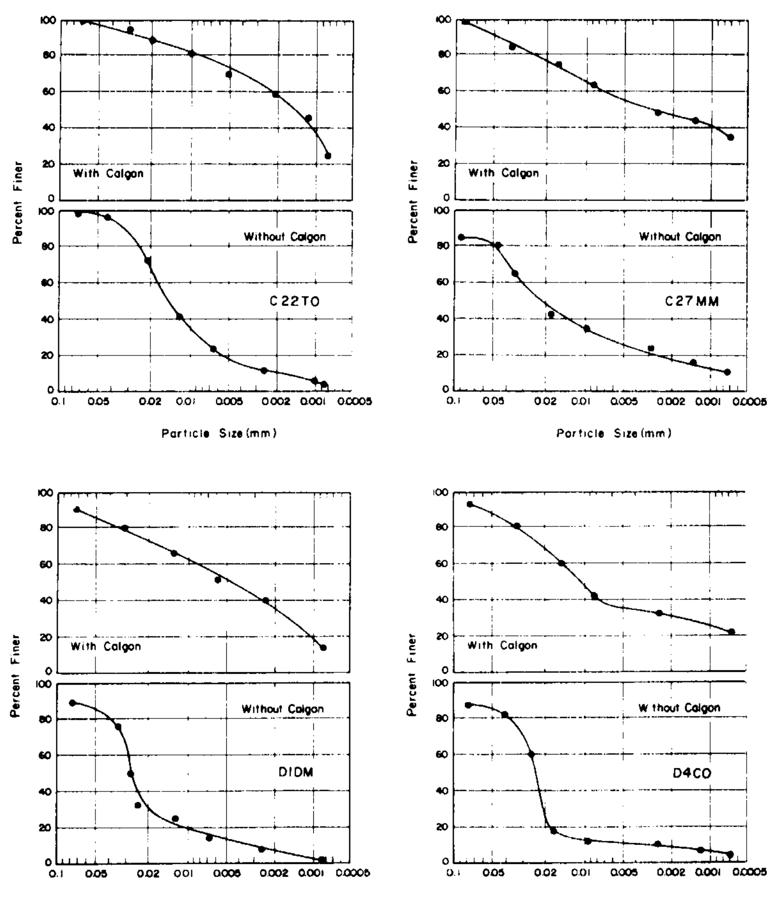

Figure 2. Particle Size Distribution Curves

most of the dredgings tested were found to be relatively well-graded, fine-grained materials.

EXPERIMENTAL PROGRAM

The permeability and drainage characteristics of several typical dredgings are examined by means of a variety of test procedures and associated interpretations. Each of these are described in the following sections, and the results and conclusions deduced therefrom are summarized.

Drainage

Three series of drainage tests were conducted to study the rate of water loss from several different dredging materials due to (a) gravity only, (b) gravity plus vacuum with the air pressure being directly applied to the surface of the dredgings, and (c) gravity plus vacuum with a membrane between the surface of the dredgings and the atmosphere. The first and second series used essentially the same equipment and procedures, with the second series incorporating several minor improvements based on the experience gained in the first series; in the third series, only gravity drainage was used.

Equipment and Procedures -- The test equipment consisted of a series of plexiglass cylinders (7 1/2 or 15 cm in diameter and 50 or 60 cm high) with porous stones fitted into the bottom of each cylinder; the effluent was collected in bottles, and the bottle was either vented to the atmosphere for gravity drainage or connected to a vacuum line for vacuum drainage. Two variations of vacuum drainage were used; in the first the air pressure due to the atmosphere was applied directly to the surface of the dredgings, whereas in the second a membrane was placed between the dredgings and the atmosphere. Although all tests were conducted in a humid room to minimize incidental water losses, it was found during the first test series that significant moisture was still being lost through the top surfaces; hence, one improvement in the second and third series was to use a smooth-fitting plexiglass disk for the top plate. The adequacy of this arrangement was proven by showing that the water losses from a cylinder filled with only water and no drainage allowed were insignificant. The test data also revealed significant losses of water due to evaporation from the collection bottles which were under vacuum, but losses from the bottles vented to the humid room atmosphere were insignificant. Corrections were made for these losses by using the data obtained from weight changes in a bottle of water on the vacuum line, but not connected to a specimen. Samples of the drainage water were taken for chemical analysis to determine the pollutants which are translocated with the effluent.

Analysis of Results -- Figure 3 gives plots of the cumulative weight of the water lost as a function of time for five typical dredgings (C2DM, C5CO, D1DM, D3TO, and D4CO) drained under three different conditions; the C samples are from the first series, whereas the D samples are from the second series. The discontinuity in the drainage rates for the C samples is attributed to a continually worsening leak in the vacuum line; when fixed on the ninth day of the test, the time rate of drainage accelerated greatly for a few days. A study of the data presented in this figure indicates that (a) the use of a vacuum extracted significantly larger quantities of water from the dredgings than did gravity alone and (b) the major effect of the vacuum occurs during the initial time period. For example, the drainage amounts after one day with vacuum were from one and a half to five times the amounts obtained for gravity alone; viewed differently, the amount of drainage with vacuum during the first five hours was about the same as by gravity alone for the first day or two, while the drainage by vacuum for one day was comparable to that by gravity alone for about a week. After three weeks, the amounts of drainage by gravity alone were about one-half of those achieved by vacuum for all samples. The results of the drainage tests for Samples D1DM, D3TO, and D4CO were used as follows to compute the coefficient of permeability, and the results are summarized in Table 3. The average flow rate, q, was determined from the total amount of water drained in the first 9 days; then, with a knowledge of the sample height and the height of water, the gradient, i, was calculated, and the permeability was computed by use of the expression

$$q = k\, i\, A \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where A is the cross-sectional area of the test specimen. In the case of the vacuum tests, the vacuum was converted to an equivalent head, and the same procedure outlined above was used. Although extracting greater quantities of water in a given time, the vacuum tests employed a greater applied head and thus generated a greater flow resistance; the head for the vacuum

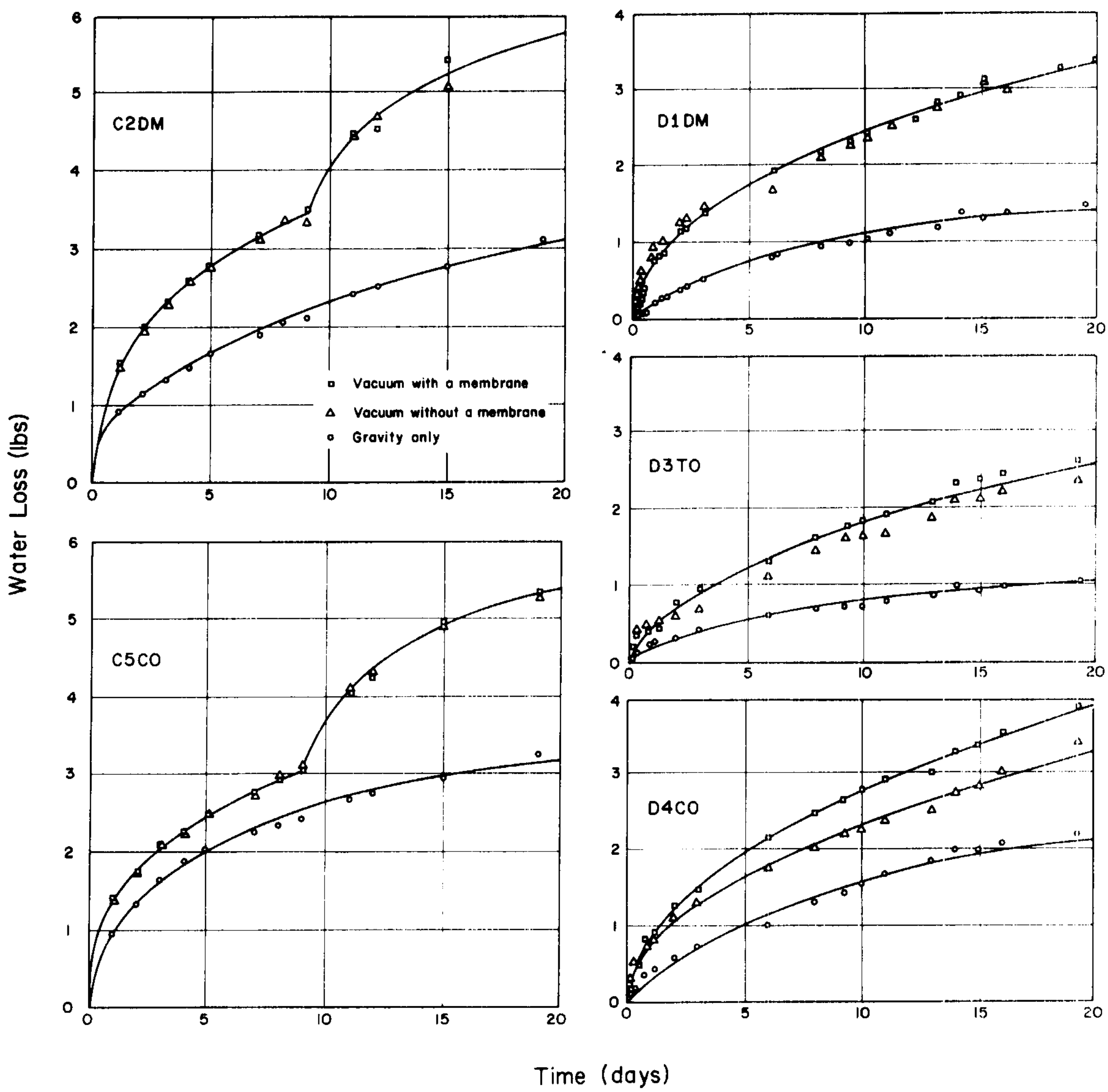

Figure 3. Rates of Drainage by Gravity and Vacuum Methods

test was about 20 times that for the gravity test, and the flow resistance was increased by a factor of about 10, as reflected in the lower values of k for the vacuum tests.

Figure 4 presents the results from six tests in the third series, which employed gravity drainage only; the top part of the figure describing each test gives the changing levels of the sediment and water surfaces as a function of time, and the lower part of each figure gives the weight of the water lost. An examination of the height-versus-time curves presented in this figure reveals that, shortly after the test was begun, the material settled and left a zone of relatively clear water above it. As drainage occurred, the sediment densified, and its top surface lowered initially more rapidly than the water surface. At a later stage in the drainage process, the surface of the sediment tended to stabilize as the water continued to drain from the specimen, until eventually the water surface reached the sediment surface. All four materials tested in this series had an initial thickness of about 18 inches (45 cm), but the time for drainage varied considerably for the different materials and with different concentrations of solids for a given material. The drainage times varied from about one day for sample C3TO at a water content of 2500% to periods in excess of 100 days. Several of the tests were concluded before all the free water had drained (Sample C3TO with an initial water content of 1100%, and both tests on Sample C22TO). Permeability determinations were made for these samples, and the results are plotted in Figure 5, where it is seen that the permeability decreases rapidly with decreasing void ratio. Since these tests extended over longer periods of time, it was possible to approximate the permeability at points throughout the drainage process; an incremental flow quantity, q, was averaged over a short period of time, and this value, together with the head and length of flow path associated with that time, were used with Equation (1) to compute the coefficient of permeability. The steeper curves are found for the samples with the lower initial concentrations of solids, thereby indicating a rapid drop in permeability with decreasing void ratio. The range of permeabilities extended from about 10^{-4} to 10^{-6} cm/sec, with the higher values being obtained at the beginning of the test and the lower values at the end. The thicker material (Sample C2DM) had the smallest change in void ratio during the test, but it still ended with the lowest void ratio of any sample in the test series.

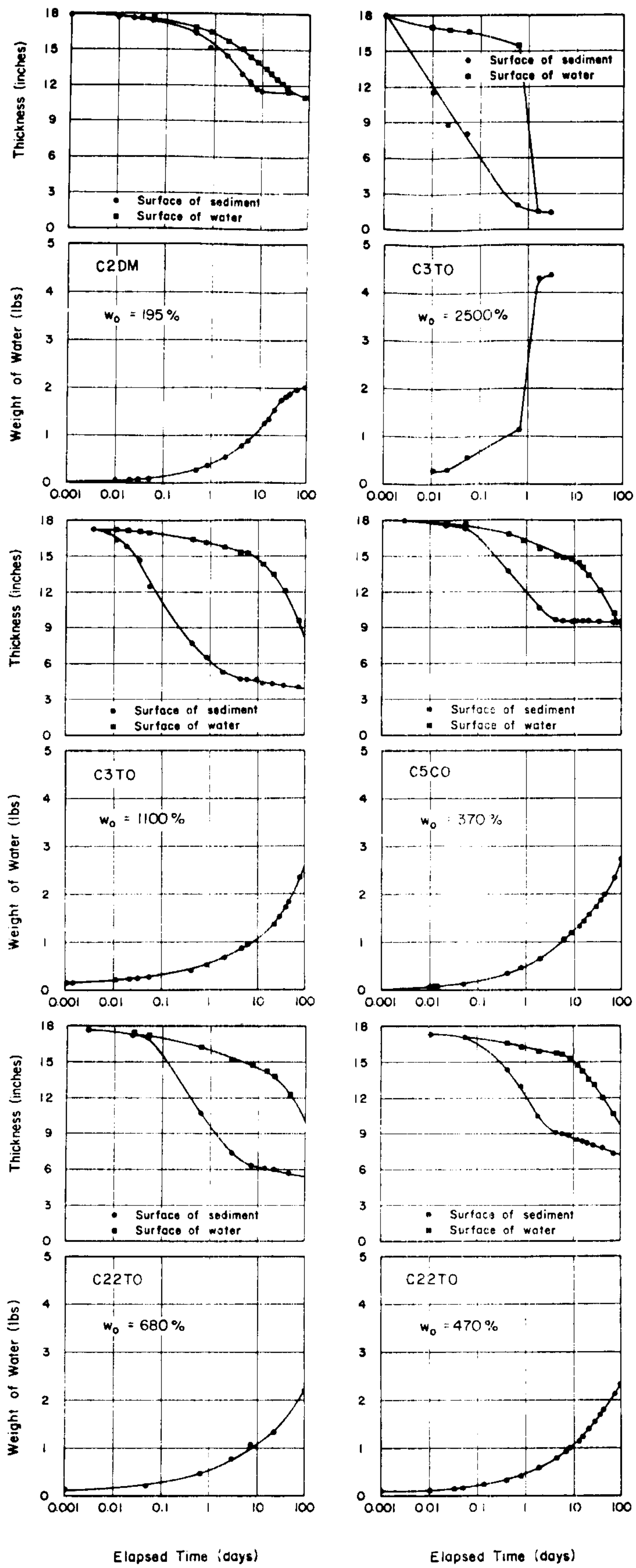

Figure 4. Data from Gravity Drainage Tests

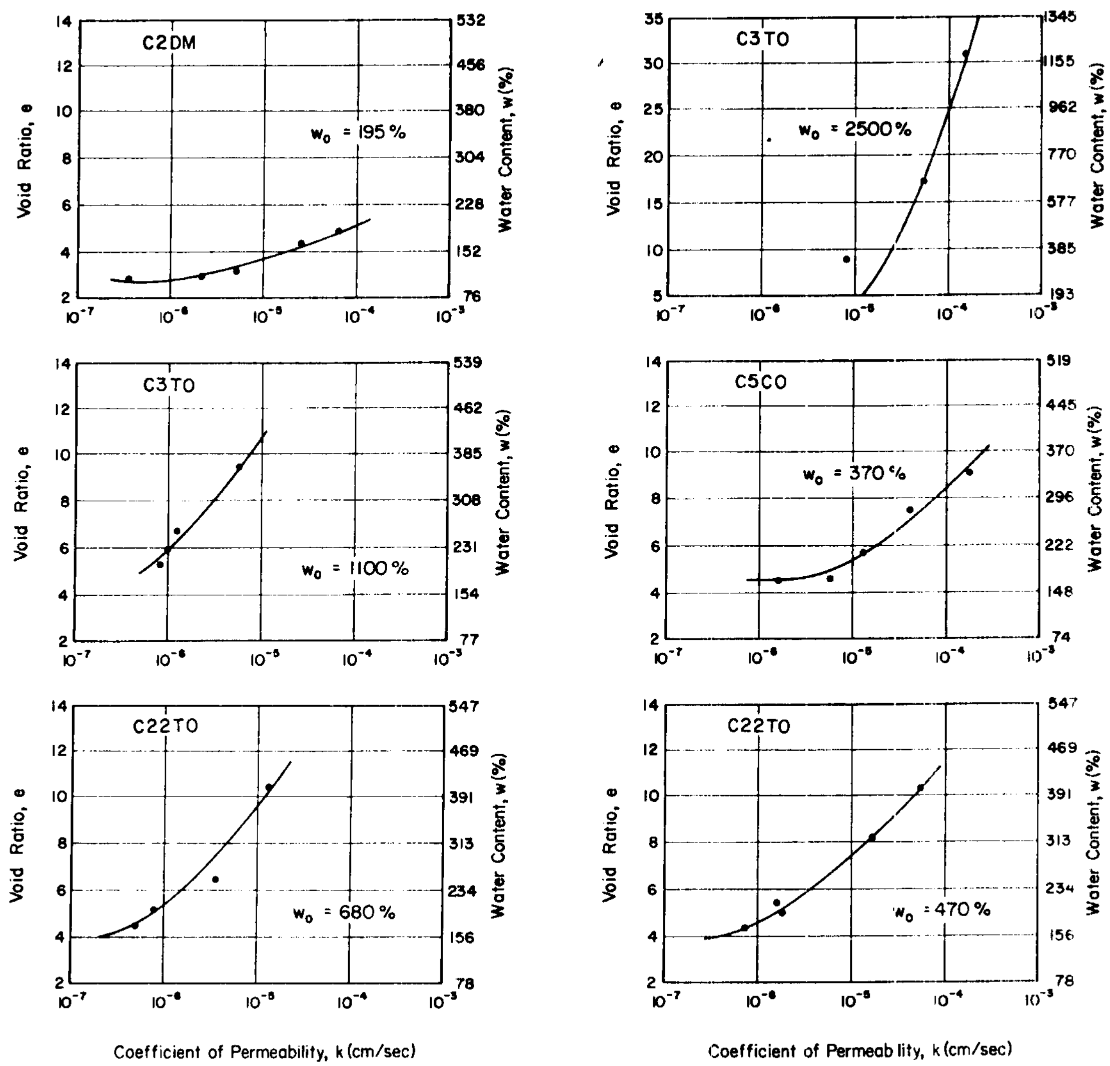

Figure 5. Permeability as a Function of Void Ratio or Water Content

The results of the chemical tests are summarized in Table 4, wherein the symbol M designates vacuum with a membrane; V, vacuum without a membrane; and G, gravity only. The fluids were drained from Samples D1DM, D3TO, and D4CO, and reported results are on a wet-weight basis. A comparison of the concentrations of the various constitutents with those of the original samples of the same materials and with various other water samples indicates that the fluids from these drainage tests contained about three times more solids than the water samples from the Maumee River and most of the other parameters, such as turbidity, biological and chemical oxygen demand, nitrogen, phosphates, sodium, potassium, and calcium were also higher. Significant quantities of mercury, copper, aluminum, and iron were found in the drainage effluents, but most of the heavy metals were retained in the sediments.

Direct Permeability

The Anteus consolidation device (Lowe, Jonas, and Obrician, 1969) was used to conduct direct permeability tests on six dredgings. "Undisturbed" samples were trimmed to fit the consolidometer ring, placed in the device, and subjected to 80 pounds per square inch (5.6 kg/cm^2) back-pressure to achieve a high degree of saturation. After being consolidated under a 10 psi (0.7 kg/cm^2) stress for at least 12 hours, the samples were unloaded and tested to determine their permeability while still under the back-pressure. The test utilized falling head conditions, wherein the driving head fell as the head on the collecting burette rose. The initial head difference was chosen at about 1.5 feet (0.5 meters), and the seepage length was about one inch (2.5 cm). Accordingly, Table 5 gives the permeability values calculated by use of the following expression:

$$k = 0.5 \left[\ln \frac{H_1}{H_2}\right] \left[\frac{A_B}{A_S}\right] \frac{L}{\Delta t} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (2)$$

where H_1 is the initial height of water at t_1, H_2 is the final height of water at t_2, Δt equals $(t_2 - t_1)$, A_B is the area of the burette, L is the length, and A_S is the area of the sample.

Conventional Consolidation

The coefficient of permeability may be computed from conventional consolidation test data by use of the following relationship:

Table 4

Chemical Analyses of Liquids from Drainage Tests

Sample	Total Solids	Total Volatile Solids	Total Suspended Solids	Total Volatile Suspended Solids	Turbidity	Acidity	Hydrogen Ion Concentration (pH)	Oxidation-Reduction Potential (Eh)	Dissolved Oxygen (DO)	Biological Oxygen Demand (BOD)	Chemical Oxygen Demand (COD)	Organic Nitrogen (N)	Ammonia (NH_3) Nitrogen (N)	Nitrite (NO_2) Nitrogen (N)	Nitrate (NO_3) Nitrogen (N)	Total Phosphate (PO_4)	Soluble Phosphate (PO_4)
	%	%	%	%	JTU	mg/gm		mv	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm
D1DM-M	0.1681	0.0914	0.0126	0.0045	92		9.2	0.030		0.1000	0.672	0.0121	0.0356	0	0.0188	0.0061	0.0061
D1DM-V	0.1724	0.0895	0.0083	0.0054	58		9.1	0.025		0.0662	0.586	0.0157	0.0291	0	0.0131	0.0020	
D1DM-G	0.1318	0.0514	0.0124	0.0072				0.030		0.0409	0.309	0.0330	0.3359	0	0.0097	0.0017	
D3TO-M	0.1254	0.0336	0.0504		100		8.8			0.0935	0.287			0	0.0073	0.0008	
D3TO-V	0.1074	0.0336	0.0358	0.0091	130		8.5	0.025		0.0850	0.186	0.0011	0	0	0.0072	0.0035	
D3TO-G	0.0751	0.0185	0.0185	0.0035	70		9.2	0.020		0.0505				0	0.0048	0.0002	
D4CO-M	0.1952	0.0540	0.0244	0.0070	55		9.0	0.045		0.0566	0.549	0.0112	0.1537	0	0.0076	0.0033	
D4CO-V	0.1381	0.0423	0.0065	0.0025	36		8.9	0.030		0.0675	0.315	0.0190	0.2532	0	0.0078	0.0061	
D4CO-G	0.1210	0.0352	0.0188	0.0045	45		8.7	0.030		0.0456				0	0.0096	0.0041	

Sample	Grease	Hydrocarbons	Aluminum (Al)	Arsenic (As)	Calcium (Ca)	Copper (Cu)	Total Iron (Fe)	Soluble Iron (Fe)	Potassium (K)	Silica (SiO_2)	Sodium (Na)	Sulfide (S)	Cadmium (Cd)	Cyanide (CN)	Lead (Pb)	Mercury (Hg)	Phenolics
	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	mg/gm	µg/gm	µg/gm	µg/gm	µg/gm	µg/gm
D1DM-M			0.030		0.0737	0.0117	0.185		0.022	0	0.200		0		0	0.[illegible]	
D1DM-V			0.030		0.0652	0.0083	0.185		0.040	0	0.180		0		0	0.0021	
D1DM-G			0.030		0.0997	0.0061	0.115		0.038	0	0.120		0		5.[illegible]		
D3TO-M			0.040		0.1000	0.0061	0.260		0.028	0	0.050		0		4.0	0.0037	
D3TO-V			0.040		0.1040	0.0061	0.200		0.020	0	0.034		0		0	0.0014	
D3TO-G			0.030		0.0870	0.0084	0.075		0.012	0	0.034		0		2.0		
D4CO-M			0.040		0.0850	0.0093	0.390		0.063	0	0.240		0		4.0	0.0134	
D4CO-V			0.040		0.0630	0.0150	0.200		0.050	0	0.200		0		5.[illegible]	0.0033	
D4CO-G			0.015		0.0720	0.0110	0.280		0.072	0	0.150		0		4.0	0.0095	

Table 5

Coefficient of Permeability from Direct Tests

Sample	Coefficient of Permeability (cm/sec)
E78TO	9.1×10^{-8}
E80TO	1.2×10^{-8}
E64TO	1.3×10^{-8}
E70TO	2.0×10^{-8}
E59TO	6.1×10^{-7}
E62TO	2.5×10^{-8}

Table 6

Coefficient of Permeability from Single-Load Consolidation Tests

Sample	Coefficient of Permeability (cm/sec)
C22TO	3.2×10^{-7}
C27MM	5.5×10^{-7}
D1DM	2.9×10^{-7}
D4CO	9.0×10^{-7}

$$k = (a_v \gamma_w c_v)/(1 + e) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where c_v is the coefficient of consolidation, γ_w is the unit weight of water, e is the void ratio, and a_v is the coefficient of compressibility, which is defined as $\Delta e/\Delta p$. In most cases, the permeability was determined when the stress on the specimen was 0.634 kg/cm^2, and a_v was taken as the change in void ratio, Δe, for the load increment from 0.634 to 1.268 kg/cm^2, in which case the stress increment, Δp, would be 0.634 kg/cm^2. The results of tests on over 25 undisturbed piston samples of typical dredgings from two disposal sites near Toledo, Ohio indicate that virtually all values for k lie between 2×10^{-8} and 4×10^{-8} cm/sec, and there is no apparent correlation with the effective particle size (0.0004 mm $< D_{10} <$ 0.0005 mm), percent clay (30 < % clay < 50), coefficient of uniformity ($15 < C_u < 40$), or liquid limit ($45 < w_L < 85$). Similar calculations for the four remolded dredging materials tested in the electro-osmosis study under an applied stress of 0.6 kg/cm^2, but with no electrical gradient, yield the results given in Table 6; these k values are approximately one order of magnitude higher than those determined from undisturbed field samples.

Slurry Consolidation

Special equipment (Sheeran and Krizek, 1971; Salem and Krizek, 1973) was used to conduct slurry consolidation tests on samples of twelve typical dredgings. The coefficient of permeability was evaluated at consolidation pressures of 4, 8, 16, and 32 psi (27.6, 55.2, 82.8, and 110.4 kN/m^2) by use of Equation (3), and the average values and ranges are shown in Figure 6. Average values for the coefficient of permeability can be reasonably well described by the empirical expression

$$\bar{k} = \log_{10}^{-1} \; [-5.90 - 1.4 \log_{10} p] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where p must be expressed in psi and $\bar{k}$ has the dimensions of cm/sec. All of the values of k for consolidation pressures of 16 or 32 psi (110 or 220 kN/m^2) were found to be less than 4×10^{-8} cm/sec; these values compare very well with values obtained from direct permeability tests (see Table 5) at comparable values of void ratio. Another relationship of engineering interest is given in Figure 7, where the average coefficient of permeability, $\bar{k}$, for all samples is plotted versus the average void ratio, $\bar{e}$, for

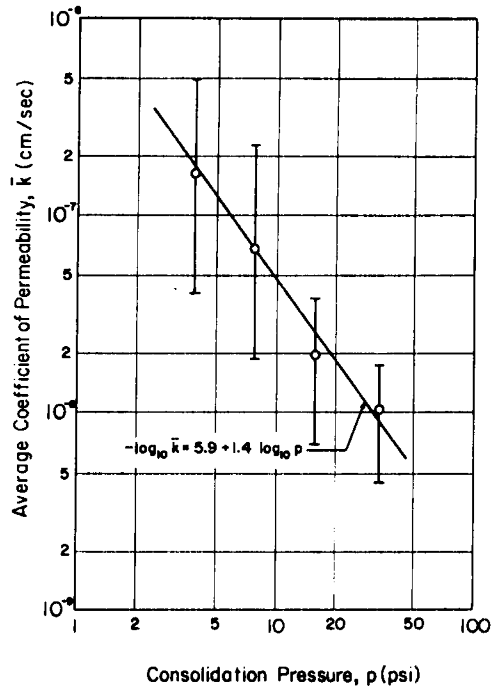

Figure 6. Average Coefficient of Permeability Versus Consolidation Pressure

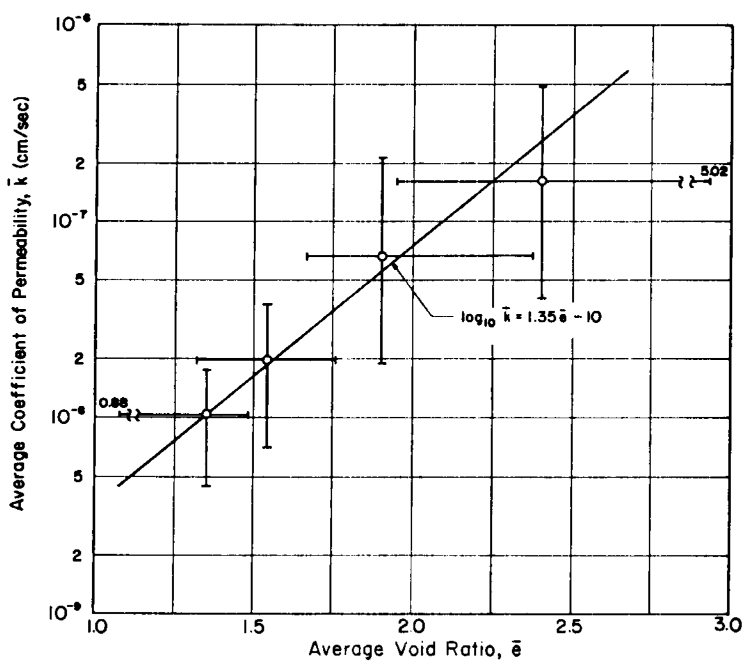

Figure 7. Average Coefficient of Permeability Versus Average Void Ratio

each load increment; the resulting equation describing this relationship can be written as

$$\bar{k} = \log_{10}^{-1} [1.35\ \bar{e} - 10] \quad \ldots\ldots\ldots\ldots \quad (5)$$

where $\bar{k}$ is expressed in cm/sec. Figure 8 shows a comparison between the permeability values determined from slurry consolidation tests and conventional consolidation tests on the slurry-consolidated blocks for two dredging samples (C10T0 and C17T0). The two types of test are mutually complementary, because they cover essentially two different ranges of void ratio; although the higher rates of change of log k with respect to e in the conventional consolidation tests may be due to the fact that a given change in void ratio causes a greater relative change in the void ratio and consequently the permeability, this probably does not account for the entire phenomenon.

Field Tests

Since the permeability of a soil mass is greatly influenced by particle size, graduation, and density (or void ratio), it will generally vary considerably in the different zones of a dredgings landfill (Giger, Franklin, and Krizek, 1973; Krizek, Soriano, and Franklin, 1974). In addition, due to the existence of sand and silt lenses and peaty material from the annual growth of vegetation, the average permeability is probably greater in the horizontal direction than in the vertical. Thus, the fill as a whole would be expected to be heterogeneous and anisotropic with respect to permeability. For this reason the data from any and all laboratory permeability tests must be viewed with caution, because it represents only discrete points in a problem which is governed more realistically by a global response. Therefore, permeability data from laboratory tests must be regarded simply as order-of-magnitude values to aid in planning more precise laboratory and/or field permeability testing programs and to facilitate preliminary calculations for drainage schemes.

In an effort to shed some light on the in-situ field permeability of a spoil area, two infiltration tests were conducted at Riverside Site in Toledo, Ohio; the majority of the dredgings in this disposal area were placed in the late 1960's and 1970, and the deposit is about 12 feet (4 meters) deep. The water table is about 0.4 to 0.8 meters below the surface, and wells with a perforated tip surrounded by a sand filter were

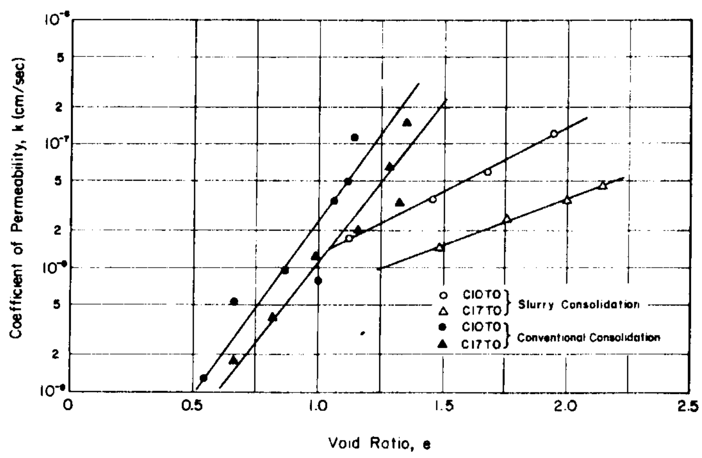

Figure 8. Comparison of Permeability Values from Slurry and Conventional Consolidation Tests

placed approximately 2 meters deep. The wells were pumped dry and the time rate of recovery was measured; data were interpreted according to the procedure outlined by Lambe and Whitman (1969; pp 284-5, Case G) with the assumption that the coefficient of permeability is isotropic, and the results indicate k values of approximately 10^{-4} cm/sec for the well located in about the middle of the disposal area and approximately 10^{-5} cm/sec for the well near the outflow weir. These values are both consistent with engineering reasoning. First, both values are significantly higher (three orders of magnitude) than those obtained from laboratory tests on samples from essentially the same locations; the probable reasons for the observed discrepancies are suggested above and probably are due in large part to the seasonal stratifications associated with dormant periods between dredging seasons. And second, the k value determined from the well near the outflow weir is about one order of magnitude smaller than that obtained from the well in the middle of the disposal area; this may be readily explanied by the fact that the finer materials tend to accumulate near the overflow weir. Reasonable (within possible experimental error) modifications in the parameters used in the above described interpretation procedure (for example, the length of the filter at the well tip) were found to change the computed permeability values by a maximum of about 25 percent, and the assumption of an anisotropic coefficient of permeability increased the calculated values by about 100 percent for $k_h = 10\ k_v$; hence, the orders of magnitude for the measured permeability values are not affected by such variations in the interpretation procedure.

Electro-osmosis

A series of electro-osmosis tests with average electrical gradients of 0, 1/2, and 1 volt per centimeter and a pressure gradient of approximately 0.1 $kg/cm^2/cm$ was conducted on four dredging samples (Krizek, Gularte, and Hummel, 1973). The grain-size distributions for the materials tested are given in Figure 2, and the average results, evaluated at either 10, 20, or 40 hours after the test was begun, are summarized in Table 7. The electro-osmotic coefficient of permeability, k_e, was determined from the relationship

$$k_e = q\ /\ \bar{v}\ A \quad . . . . \quad (6)$$

Table 7

Average Electro-osmotic Coefficient of Permeability and Flow Efficiency

Sample	Time (hours)	Electro-osmotic Coefficient of Permeability, k_e (cm^2/volt-sec)	Flow Efficiency	
			milliliters / watt-hour x liter	gallons / watt-hour x cu. yd.
C22TO	20	1.49×10^{-5}	17.9	3.62
	40	1.31×10^{-5}	17.9	3.62
C27MM	20	2.70×10^{-5}	24.6	4.97
	40	2.02×10^{-5}	23.1	4.66
D1DM	10	4.54×10^{-5}	30.4	6.14
	20	3.30×10^{-5}	27.4	5.54
D4CO	10	3.69×10^{-5}	13.0	2.62
	20	2.78×10^{-5}	12.9	2.60

Table 8

Comparison of Dewatering Rates With and Without an Electrical Gradient

Sample	Time	Electrical Gradient			
		None		1 volt/cm	
		Total Flow (liters)	Flow from Bottom (liters)	Total Flow (liters)	Flow from Bottom (liters)
C22TO	6 days	0.91	0.48	1.15	0.96
	20 hours	0.50 (55%)	0.24 (50%)	0.60 (52%)	0.43 (45%)
	40 hours	0.71 (78%)	0.35 (73%)	0.89 (77%)	0.71 (73%)
C27MM	6 days	0.89	0.45	1.17	1.07
	20 hours	0.56 (63%)	0.28 (63%)	0.73 (63%)	0.64 (60%)
	40 hours	0.76 (85%)	0.38 (85%)	1.05 (90%)	0.95 (89%)
D1DM	6 days	0.59	0.31	0.94	0.80
	10 hours	0.49 (83%)	0.25 (81%)	0.67 (71%)	0.54 (67%)
	20 hours	0.57 (97%)	0.30 (97%)	0.88 (94%)	0.74 (93%)
D4CO	6 days	0.94	0.48	1.10	0.94
	10 hours	0.59 (63%)	0.29 (60%)	0.69 (63%)	0.54 (57%)
	20 hours	0.83 (88%)	0.42 (88%)	0.98 (89%)	0.82 (87%)

where q is the time rate of fluid flow, A is the cross-sectional area perpendicular to the direction of flow, and $\bar{v}$ is the average voltage gradient across the sample, and the results range from about 1.5×10^{-5} to 4.5×10^{-5} cm^2/volt-sec; these values are of the same order of magnitude as those which characterize a large variety of soils, namely, 5×10^{-5} cm^2/volt-sec.

By using cumulative flow quantities for the indicated times and defining the energy consumed during the associated time interval as the area under the current-time curve multiplied by the applied voltage, we can determine the effectiveness of the electro-osmotic treatment; as seen in Table 7, the quantity of water removed from each of the four materials varies from about 3 to 6 gallons per watt-hour per cubic yard (or 15 to 30 milliliters per watt-hour per liter) of material. However, the energy consumption increases sharply for time periods longer than those selected; for example, Table 8 shows that approximately 75 to 95 percent of the total flow after 6 days occurs in the selected time periods, and additional treatment produces little additional drainage. Values are given in Table 8 for total flow and bottom flow because the specimens were drained at both the top and the bottom, but the electrical gradient was applied in one direction only; hence, in one half of the specimen the pressure gradient and the electrical gradient oppose each other, whereas in the other half they are additive. A good check on experimental reliability can be obtained by comparing these two flow values for the case of no electrical gradient; the total flow quantities should be double the flow quantities from the bottom only, and this is approximately true.

Although certain similarities exist between fluid flow due to an electrical gradient and fluid flow due to a hydraulic pressure gradient, the physics of the situation suggests that there are substantial differences. For example, in fluid flow due to a hydraulic gradient, various soluble ions may translocate with the pore fluid and leave the soil as it is drained, or they may be leached from the soil as fresh water becomes available from precipitation; however, in fluid flow due to an electrical gradient, the electrical current causes the cations to migrate toward the cathode, and they carry the water with them. In the latter process the soil is depleted of its cations much more rapidly than in the former process, and the major effect is noticed quickly in a soil subjected to electro-osmotic treatment. Tests on the effluents extracted from the dredgings indicated that the electro-osmotic treatment removed substantial amounts of soluble ions from the specimen with the quantity increasing as the electrical gradient increased, but the heavy metals remained in the sediments, regardless of the degree of the electrical treatment.

POTENTIAL INFLUENCE OF ORGANIC CONTENT AND GAS GENERATION

Little is known regarding the effect of organic content on the permeability of a soil. Some evidence (Arman, 1970) indicates that, if organic matter of a fibrous nature is present in a clay or silt, the coefficient of permeability, k, increases rapidly until the organic content reaches about 40 percent, after which there is not much additional change. However, exactly the opposite happens for sands, with the coefficient of permeability stabilizing again at about 40 percent organic content. In a plot of permeability versus organic content, the curves for sand, silt, and clay converge at around 40 percent organic content to a constant value -- the sand from above and the clay and silt from below -- that seems to be the coefficient of permeability of the organic matter alone. Although the above behavior was reported for the case of fibrous organic matter, there was no data concerning the effect of finely divided organic matter on the permeability (that is, organic matter with a particle size similar to silt or clay). However, organic matter in dredgings may have a more complex incluence on permeability, depending upon the degree of humification and the chemical composition of the organic matter. While fibrous, peaty material may provide an open, porous, permeable soil, fine-grained organic matter may adsorb water and swell, thereby blocking the pore passages and impeding the flow of water through the soil. In a similar fashion, fats, grease, and oil may tend to obstruct the pore passages in a fine-grained soil.

The degree of saturation is known to exert a large influence on the permeability of a given soil. Since many of these polluted dredgings are actively decomposing various organic compounds and generating gases in the process, the entrapped gases tend to block pore passages and thereby constitute a complex and influential factor in the permeability of these materials. In some cases the response for the low intensity of loading in the slurry consolidation tests show an increasing void ratio with time, thereby indicating that the pressures developed by the generated gases exceed the applied pressures and cause an expansion of the sample; however, the more regular behavior of the higher loadings suggests that gas generation may decrease with time and be suppressed at higher loadings.

SYNTHESIS OF RESULTS

As summarized in Figure 9, values for the coefficient of permeability were found to range from about 10^{-4} to 10^{-9} cm/sec, depending primarily on the void ratio of the specimen (which is, in turn, related to the type of test). The dredgings with the highest void ratios ($2 < e < 10$) were tested in the drainage tests, and permeability coefficients on the order of 10^{-4} to 10^{-6} cm/sec were determined; for the drainage tests in which a partial vacuum was applied (thereby consolidating the sample somewhat and decreasing the void ratio), k values on the order of 10^{-6} to 10^{-7} cm/sec were obtained. Void ratios in the slurry consolidation tests ranged from about 1.3 to 2.4, and the resulting coefficients of permeability were approximately 10^{-7} to 10^{-8} cm/sec. The direct permeability tests, the single-load consolidation tests, and the conventional consolidation tests were conducted on samples with void ratios on the order of 0.6 to 1.6, and the associated permeability values lie between 10^{-7} and 10^{-9} cm/sec with most on the order of 10^{-7} to 10^{-8} cm/sec. As expected, the in-situ field tests yielded much higher values for the permeability (10^{-4} to 10^{-5} cm/sec); this was probably due in large part to the seasonal stratifications associated with dormant periods during the dredging seasons. Finally, the electro-osmotic coefficient of permeability (not plotted in Figure 9) was found to range from about 1.5×10^{-5} to 4.5×10^{-5} cm^2/volt-sec, and this value is consistent with that found for a large variety of soils.

SUMMARY AND CONCLUSIONS

Based on the mineralogy, organic content, Atterberg limits, and gradation of the dredgings tested, they may be classified as fine-grained, organic soils (OH, according to the Unified Classification System), and their permeability and drainage characteristics are generally consistent with those associated with these types of soils. However, even within this framework, substantial differences may be observed in permeability values and drainage behavior, especially between results obtained from laboratory and field tests. In an effort to address this problem as comprehensively as possible, various types of independent laboratory tests and two field

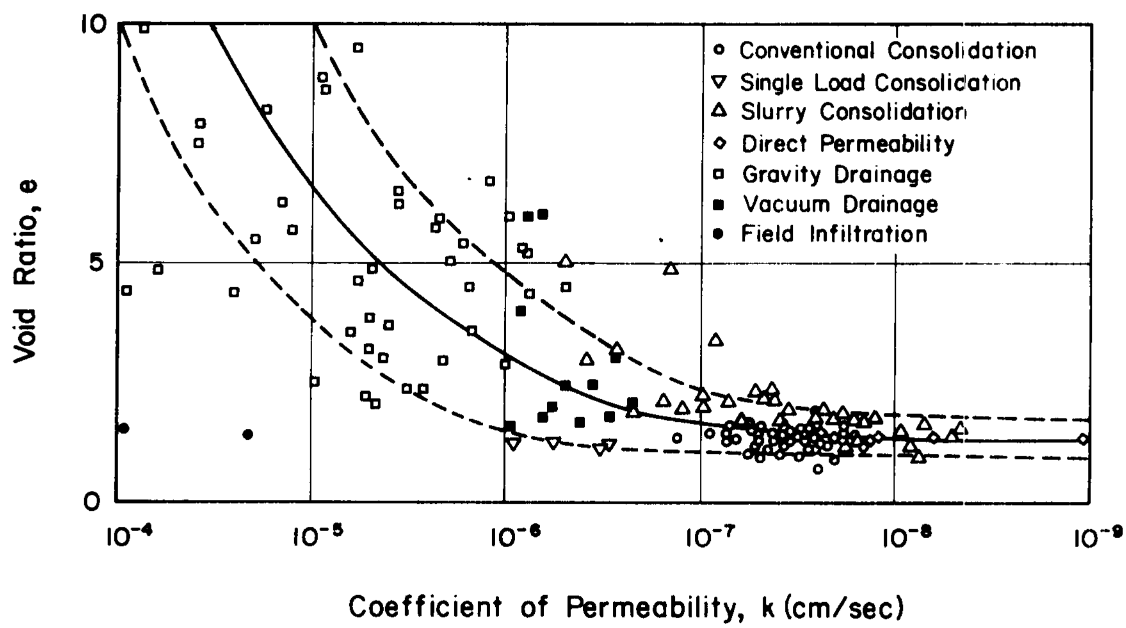

Figure 9. Summary of Values for Coefficient of Permeability

infiltration tests were conducted, and the results from each type of test were evaluated in accordance with standard procedures.

The drainage characteristics of a given material were found to depend on the nature of the solids and the fluids as well as the water content at the time of drainage. Different dredgings drain at different rates and are affected to different degrees by the application of a partial vacuum. In general, vacuum drainage was found to remove water from dredgings much faster than gravity drainage alone, and it allowed greater amounts of water to be extracted in a given period of time. However, the maximum effect of vacuum on the drainage response was achieved during the initial time period, and less significant effects were observed over longer periods of time.

The coefficient of permeability is strongly dependent on the void ratio and decreases from about 10^{-4} to 10^{-9} cm/sec as the void ratio decreases from approximately 10 to 1. Most permeability values for the firmer materials, which had void ratios on the order of 1 to 2, were in the range of 10^{-7} to 10^{-8} cm/sec. Field infiltration tests yielded permeability coefficients approximately three orders of magnitude higher than those obtained from laboratory tests on undisturbed and remolded samples with comparable void ratios. The electro-osmotic coefficient of permeability was found to be about 3×10^{-5} cm^2/volt-sec, which is approximately one-half the value determined for a large variety of soils.

ACKNOWLEDGMENT

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CONCEPTS FOR THE RECLAMATION OF DREDGED MATERIAL

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INTRODUCTION

Over the past two decades, increasing use has been made of containment facilities to store the materials generated in dredging operations. For a number of reasons, land disposal of dredged material is expected to increase and it has been estimated that 7,000 acres of new land will be required each year for the containment of material generated from maintenance dredging operations by the Corps of Engineers. Because of intensive land use in areas adjacent to many dredging operations, suitable land disposal sites will be difficult to obtain in the future.

To date, land disposal of dredged material has used conventional containment facilities to separate the solids and provide long-term, if not indefinite, storage of the separated material. In order to reduce the commitment of land for this purpose, investigations were made of alternatives to the use of conventional containment facilities. Concepts for reducing the size and for prolonging the useful life of containment facilities were defined for evaluation purposes. These included the separation and reclamation of useful materials, the separation and handling of materials for offsite disposal, and methods for improving the effectiveness of separation and handling processes.

This work was performed as part of the U.S. Army Corps of Engineers, "Dredged Material Research Program" under a contract study initiated by Hittman Associates in June 1973. The report covering this work is now being published. The contract was subsequently amended to include the evaluation of the performance of silt separation basins. This paper incorporates certain of the results of this later work.

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STUDY PARAMETERS

The majority of current dredging is performed using hydraulic dredges. In many cases where mechanical hopper dredges are used, the materials are fluidized for further handling either in the dredge hoppers, barges or from basins in which the material is dumped for temporary storage. For these reasons, the studies of alternative containment concepts were based on receiving the dredged material as a slurry. Because of the nature of maintenance dredging operations, it was assumed that the slurries would be relatively dilute with solids concentrations in the range of 10 to 20 percent by weight.

Material Characteristics

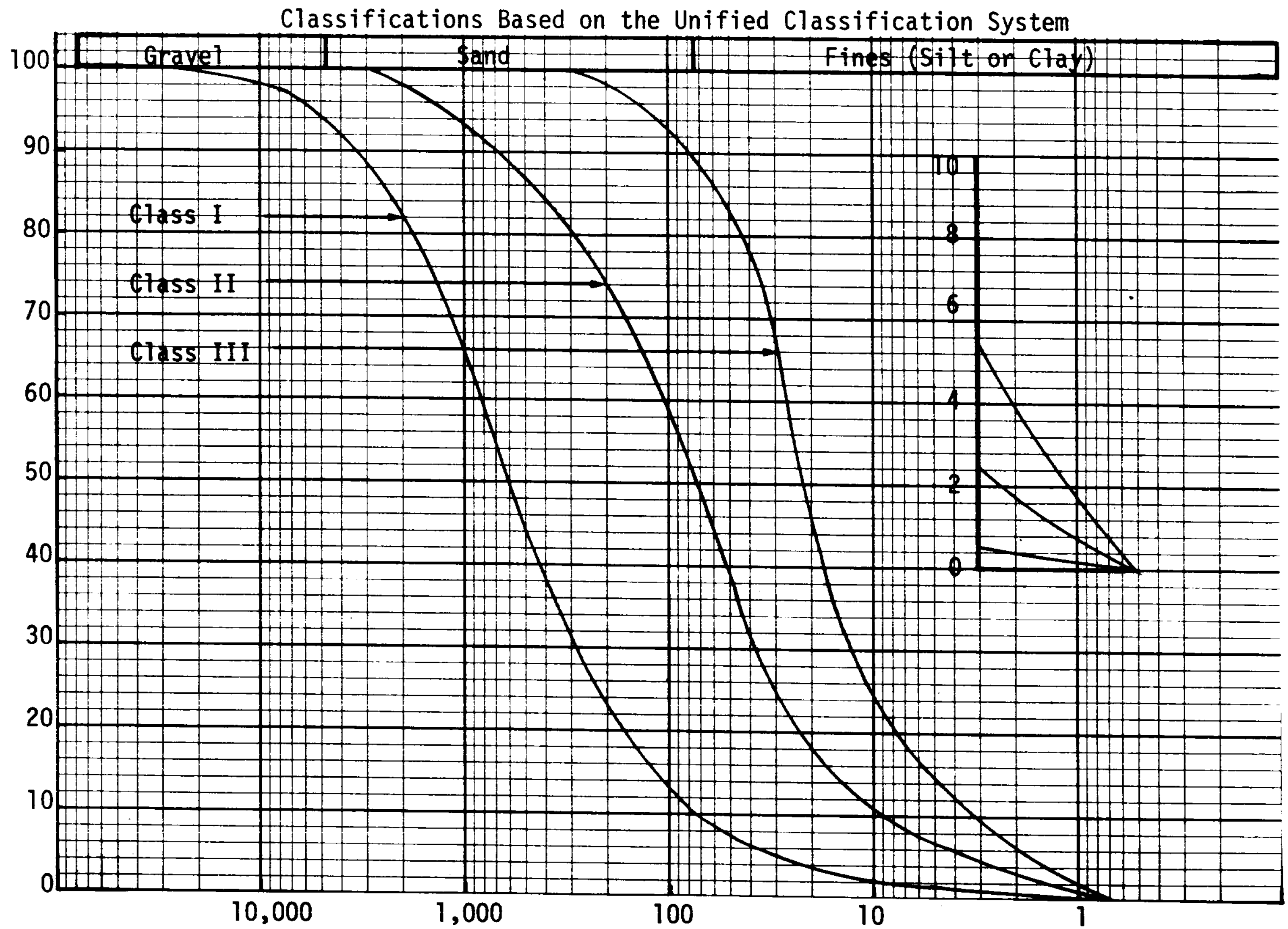
The composition of dredged material varies widely by geographical area because of the land use, soil and runoff characteristics of the surrounding areas. The composition of the dredged material will also vary due to the hydraulic characteristics of the body of water in which they are deposited. Recognizing that such variations exist, typical particle distributions were assumed for the purposes of these studies. These are shown on Figure 1. The three classes of material are considered to be typical of:

Class I	Flowing rivers and glacial deposits
Class II	Waters with moderate flow or tidal action
Class III	Relatively still lakes

Dredge Flow-Rates

Maintenance dredging is performed by dredges of various types and sizes depending upon the material to be dredged, the depth of the deposits, the transport distance and other factors. The hydraulic dredges used on these projects vary in size from 8 to 36 inches.

The pumping rates for the dredges will also vary depending upon the material to be dredged and the size of the discharge line. For eight-inch lines, the minimum velocity required for the transport of slurries will range from 6.8 feet per second for fine sand to 7.9 feet per second for medium sand, 9.0 feet per second for coarse sand and 11.25 feet per second for gravel. For larger lines, the velocity must generally be increased by the ratio of the square root of the diameter. As an example, the required velocity for a 32-inch dredge would be twice that of an eight-inch dredge, i.e., $\sqrt{32/8}$. Figure 2 shows the flow rates typically



TYPICAL GRAIN SIZE DISTRIBUTIONS FOR DREDGED MATERIAL

Figure 1

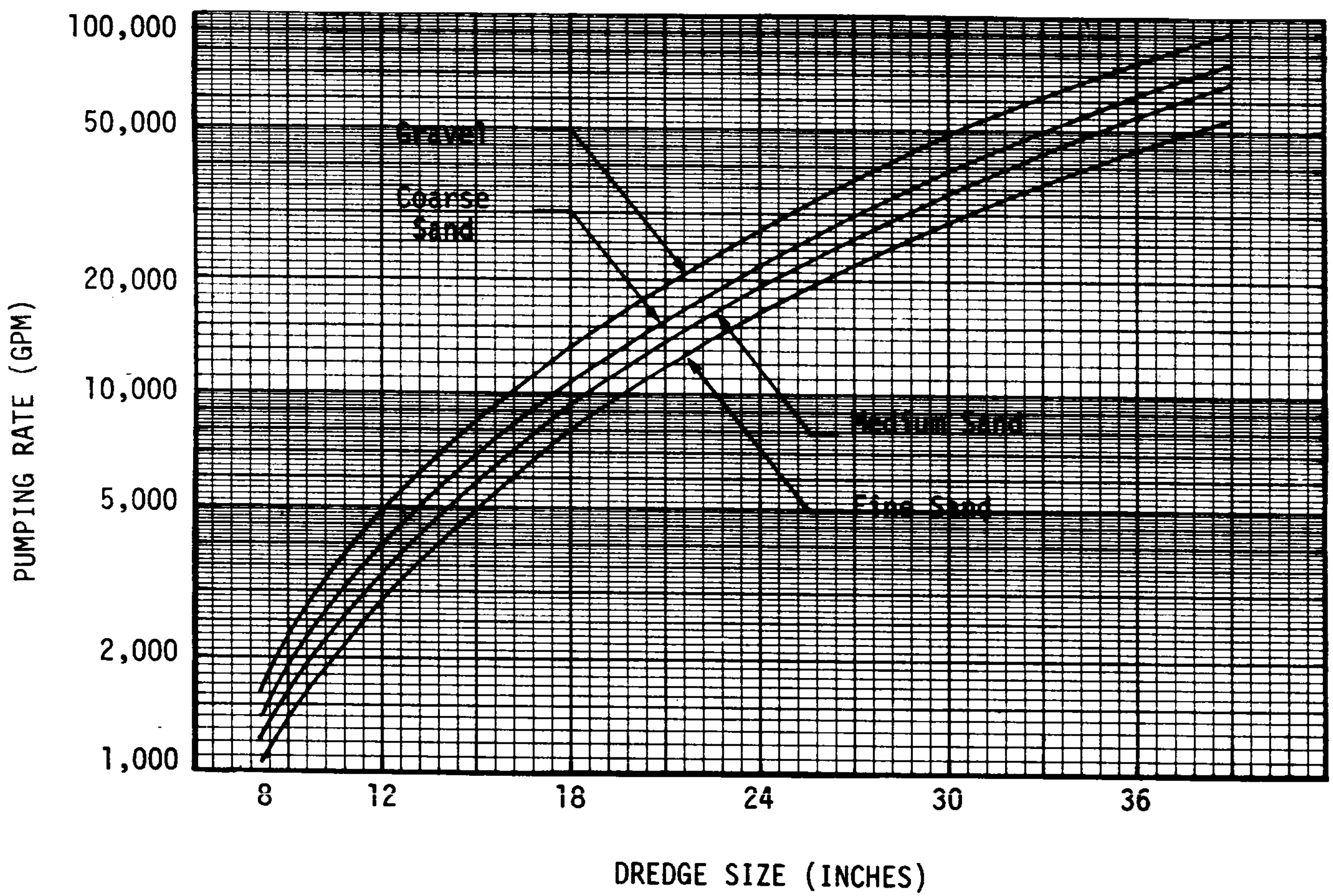


Figure 2. Dredging Rates to Transport Various Materials

required to transport various materials. The flow must be adequate to maintain the largest particles present in suspension.

Analytical Methods

The design methods for conventional containment basins vary widely and there is no generally accepted practice. For this reason, it was necessary to adapt methods used in the design of settling tanks and clarifiers to predict the performance of conventional containment basins and separation systems. These methods are outlined below.

An equation was derived to define the required removals to meet various water quality requirements. This equation is based on a flow and materials balance of the settling basin which yields the following:

$$R = \left[1 - \left[\frac{\frac{100}{C_1} - 1}{\frac{1000}{C_2} - 1} \right] \right] \times 100$$

Where:

R = Required solids removal (%)

C_1 = Concentration of the incoming slurry (wt%)

C_2 = Concentration of the effluent (gms/kg or ppm<1000)

The units for C_2, gm/kg, are used in lieu of gms per liter in order to simplify the equation. The difference between the two values is a few hundredths of a percent and can be used interchangeably for most purposes.

Having calculated the required removal, the minimum size particle which must be removed is determined by entering the appropriate particle distribution curve previously shown in Figure 1. The required size of the basin is determined by the following equation:

$$A = K\frac{Q}{V_s}$$

Where:

V_s = Critical settling velocity, cm/sec

g = Acceleration due to gravity = 981 cm/sec^2

D = Diameter of a spherical particle, cm

S_s = Specific gravity of the particle

V = Kinematic viscosity of water, cm/sec

Figure 3 is a graph of the settling velocities of various size particles expressed in terms of feet per second and microns for convenience of use.

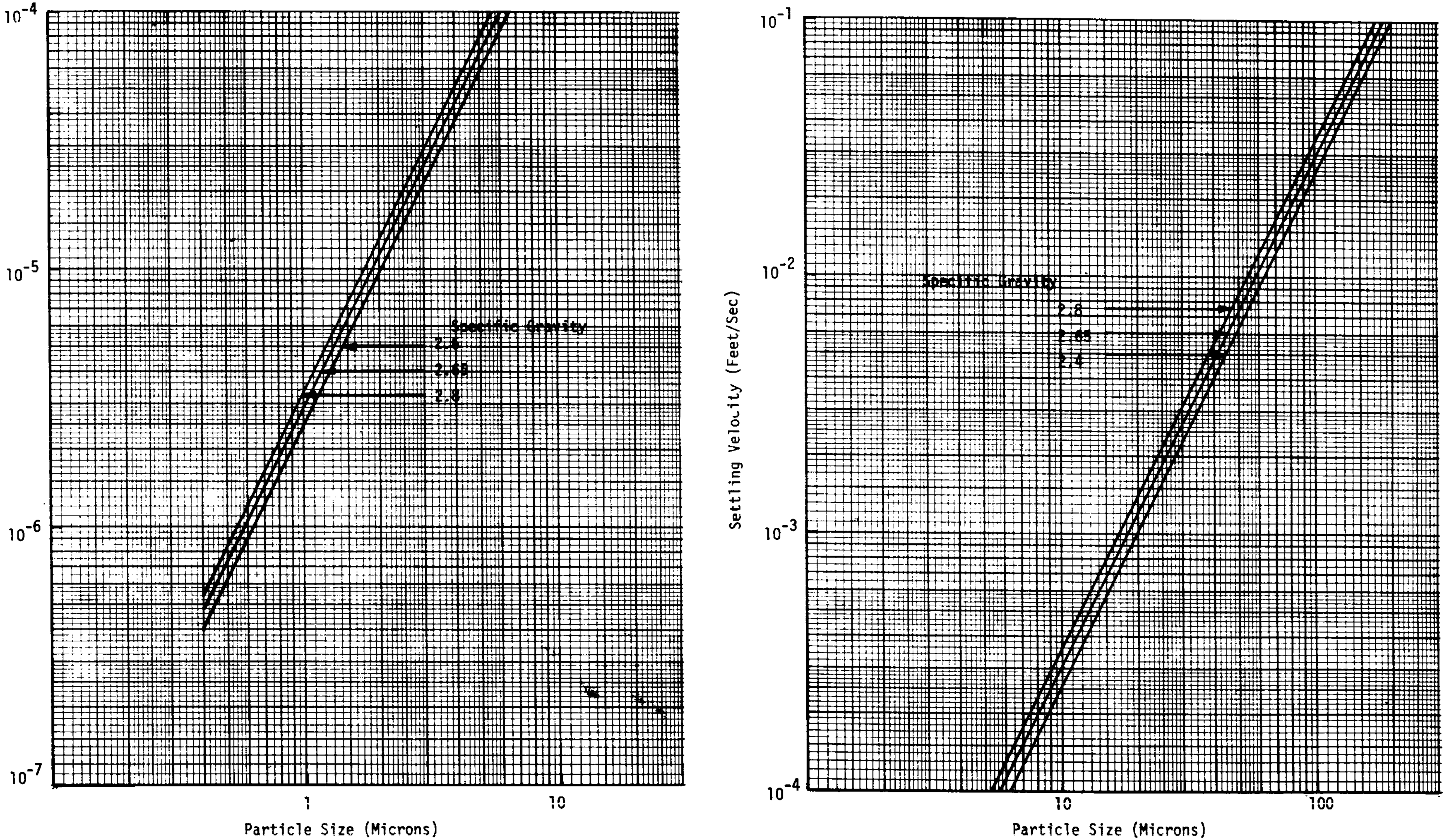
The use of overflow velocity and settling velocities determined by Stokes Law for predicting the performance of containment basins can be questioned. In field tests and in the operation of containment basins, greater retention of particles has been observed than would be predicted by these theories. It has been found that the use of particle distributions determined by hydrometer tests in which dispersants have not been used give a better indication of the settling characteristics than the particle distributions determined with the use of dispersants. The apparent reason being that many of the fine-grained particles are agglomerated and remain so during dredging. Even the non-dispersed particle distributions do not fully predict the solids retention in some cases. This is probably due to entrapment, weighting effects and other factors. In the absence of analytical methods to account for these effects, it appears that the non-dispersed particle distributions and above theories provide a conservative basis for containment basin design.

Commercially Available Equipment

In these investigations, data was collected on the commercially available equipment used by the sand and gravel and mineral industries to separate and process slurries. This included grizzlies, vibrating screens, scalpers and classifiers, hydrocyclones, spiral screw classifiers, thickeners, small dredges, etc. Where possible, the concepts were based on commercially available equipment.

DREDGED MATERIAL SEPARATION

Conventional containment facilities are constructed to serve two general purposes. First the containment basins must retain a significant portion of the suspended solids in order to provide an overflow of adequate quality to permit the water to be returned to the body of water being dredged or other receiving waters. Second, the basin must have adequate volume to store the materials to be dredged in the vicinity.



SETTLING VELOCITIES OF FINE GRAINED PARTICLES

(STOKES LAW – 68°F WATER)

Figure 3

The first condition requires basins with a large area to reduce the overflow velocity to less than the settling velocity of the particles which must be retained to achieve the required water quality. The second condition requires that the basin have sufficient depth. Where the depth is not adequate, the area of the basin must be increased to provide the required storage. This is often the case in areas where periodic maintenance dredging is required. As a result, the land area requirements become primarily a function of the storage requirement.

If it were feasible to remove the retained dredged material during or between dredging operations, it would be possible to reduce the required area of containment facilities.

Potential for Storage Reduction

The potential for the reduction in the quantities of dredged material to be stored can be illustrated for the three assumed classes of material previously shown in Figure 1.

Figure 4 shows the sand, gravel and silt fractions of these materials. As indicated, the sand and gravel fractions for the three classes of material are 90, 50 and 10 percent, respectively. After the sand and gravel is removed, the majority of the remaining material will generally be in the silt range (10 to 74 microns). For the three assumed classes of material, the silt fraction represents 70 to 80 percent of the remaining solids after sand and gravel separation. With the removal of the sand, gravel, and silt, the quantities of material which must be handled or stored are reduced to 2, 10 and 23 percent, respectively, of the original amount of solid material dredged. The benefits in terms of long-term storage of dredged material are obvious.

Separation System Options

Figure 5 is a functional diagram showing the options for dredged material separation and handling. As indicated, the dredged slurry would first undergo processing to remove the trash and other materials.

The first products to be separated would be the sand and gravel. These would undergo beneficiation in conventional equipment used in the industry. The processed materials would then be stockpiled. Off-site transport would be by truck, rail or barge as products for the construction industry.

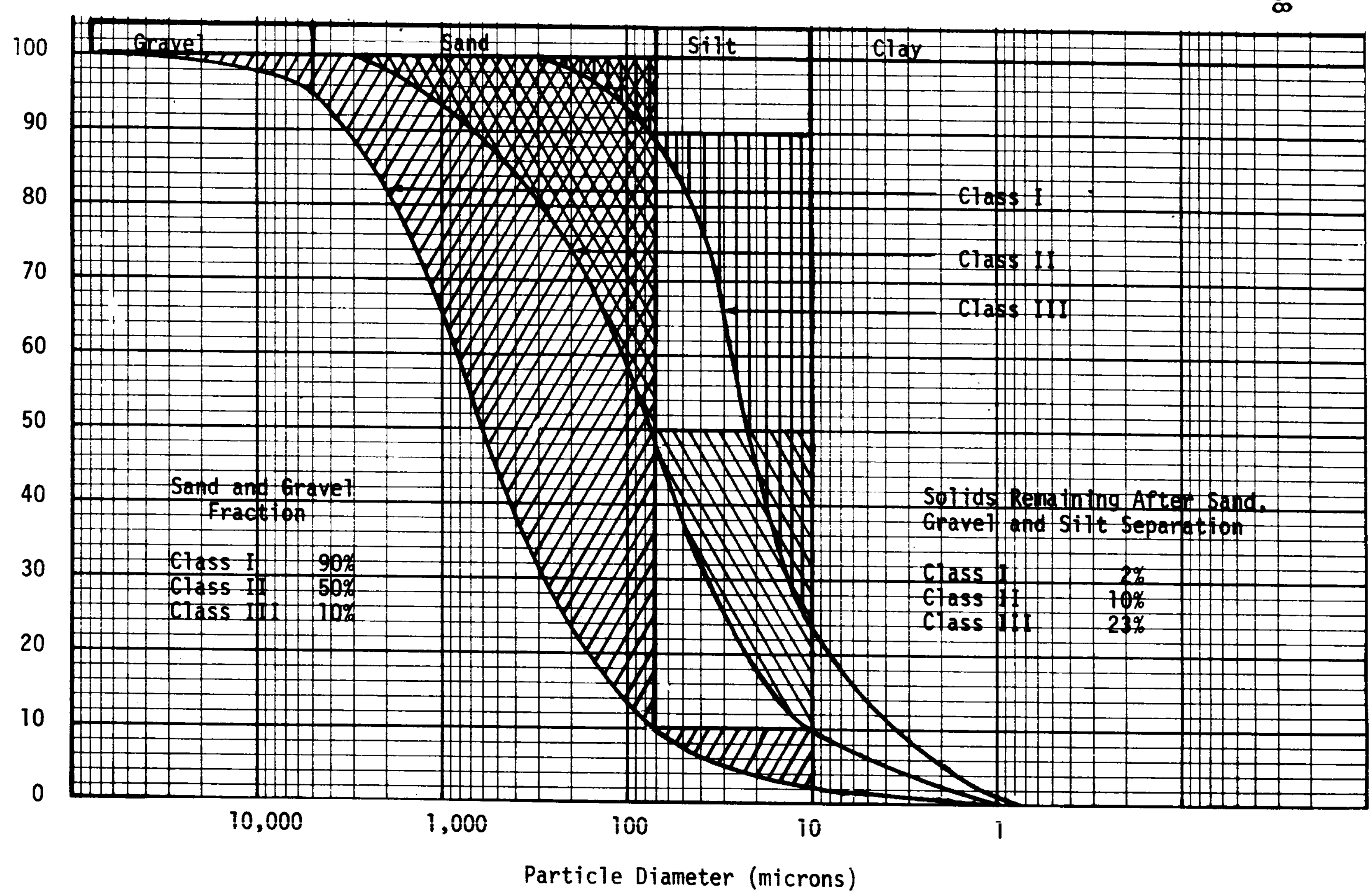


Figure 4. Solids Reduction by Gravel, Sand and Silt Separation

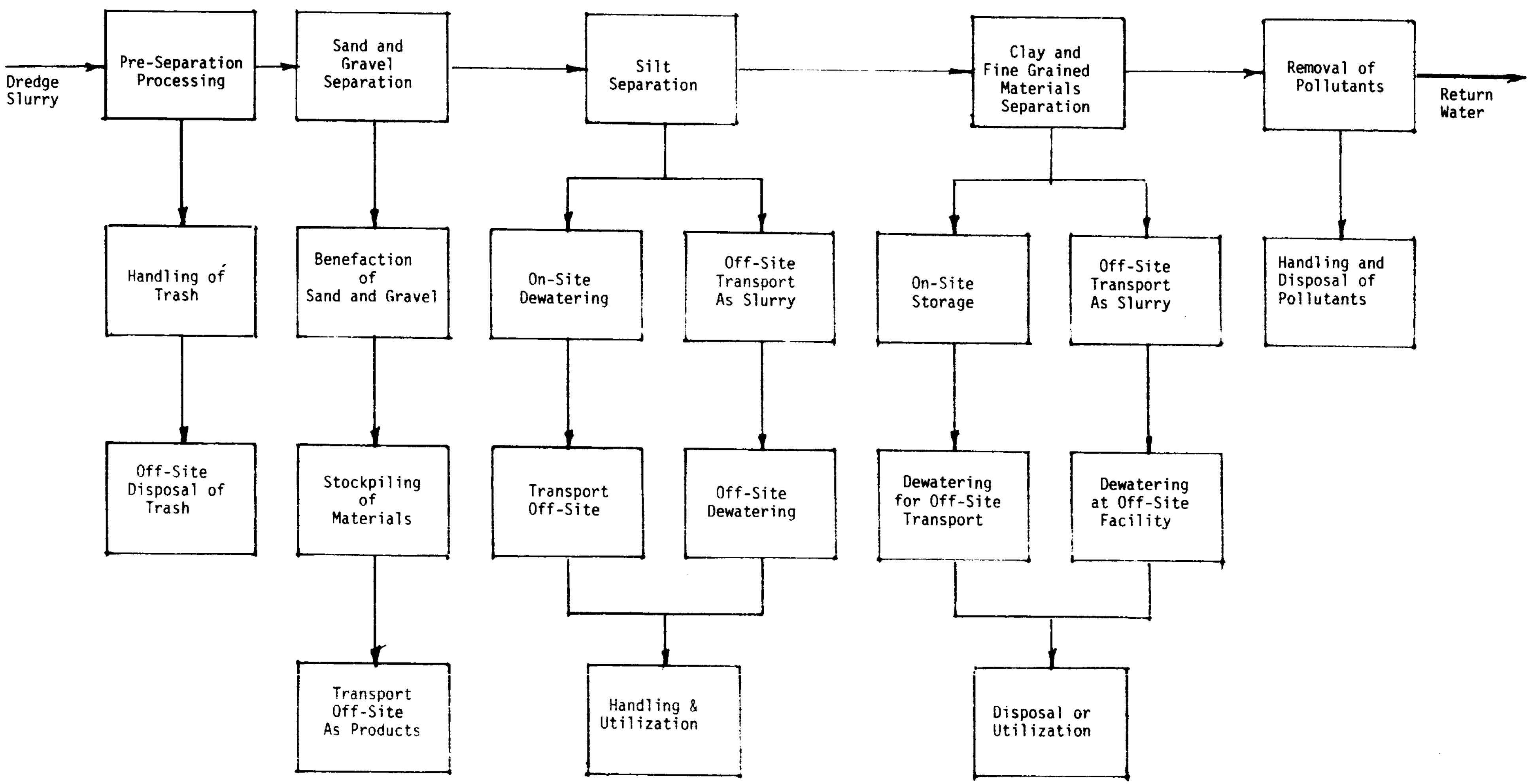


FIGURE 5. DREDGED MATERIAL SEPARATION AND HANDLING OPTIONS

The silt fraction would then be selectively removed to provide a material that is more readily dewatered than a mixture of silt and clay. The separated silt could either be dewatered on-site or transported off-site as a slurry to off-site dewatering facilities. Following dewatering, the material could be handled using conventional equipment for use as fill. Alternatively, the material could be placed hydraulically and dewatered in place.

The clay materials represent the most difficult to dewater and handle. These materials could be stored on-site in conventional containment basins. As previously indicated, the quantity of material to be stored would be greatly reduced because of the removal of sand, gravel and silt. The stored material could also be allowed to dewater for subsequent removal to off-site locations. Where off-site transport to nearby locations is desired, pipeline transport of a concentrated slurry will generally be more desirable than truck transport.

In cases where the dredged slurry contains undesirable dissolved materials, it may be necessary to further process the effluent from clay separation.

Potential Yields

The potential yields of materials will depend upon the dredging rate, the class of material, and the dredge slurry concentration. Table 1 shows the potential yields of various materials at a dredge slurry concentration of 15 percent solids.

Table 1. Potential Yields of Materials

Dredge Size (in)	Flow Rate (gpm)	Material Class	Potential Yield (TPH) Sand/Gravel	Silt	Clay	Total
8	1,500	I	56	5	1	62
		II	31	25	6	
		III	6	41	15	
12	4,000	I	144	13	3	160
		II	80	64	16	
		III	16	107	37	
18	11,000	I	406	36	10	432
		II	226	81	45	
		III	45	302	105	

Dredge Size (in)	Flow Rate (gpm)	Material Class	Sand/ Gravel	Silt	Clay	Total
24	22,000	I	920	80	20	1020
		II	510	408	102	
		III	102	683	235	
30	40,000	I	1480	132	38	1650
		II	825	660	165	
		III	165	1050	435	
36	60,000	I	2220	198	52	2470
		II	1235	988	247	
		III	247	1650	473	

SAND AND GRAVEL SEPARATION

Sand and gravel are commonly separated using hydraulic methods in the sand and gravel industry. The method is used where the material is mined using hydraulic dredges. It is also used where the material is mined dry. In this case, water is added to create a slurry.

Scalpers and classifiers are used to separate and classify the coarse, medium, and fine sands. These units are elevated, rectangular tanks with bottom hoppers to release the settled material. The released material drops into spiral classifiers or sand screws where it is agitated and worked to remove silt and clay, dewatered and elevated onto conveyors for stockpiling as a finished product.

Scalpers and classifiers are made in sizes up to 48 by 12 feet which have flow capacities depending upon the size of material to be separated. The flow rates for units of this size are typically 8000 gpm for 100 mesh material (149 microns), 4200 gpm for 150 mesh (105 microns) and 2150 gpm for 200 mesh (74 microns).

The principal differences between normal sand and gravel processing and the reclamation of sand and gravel from dredged material will be the higher flow rates, lower solids concentrations, and lower sand and gravel fractions. The dredges used in sand processing are generally small capacity (6 to 12 inches) and are operated in deposits with sand and gravel fractions up to 90 percent. Slurry concentrations up to 50 percent are delivered to the scalpers.

In the reclamation of sand from dredged material, it is desirable to reclaim the maximum amount of material to reduce storage requirements.

For that reason, scalpers would be operated to remove 200 mesh (74 micron) material and the flow rate would have to be maintained at the lower values. Although it is technically feasible to feed the dredged slurry directly to multiple scalpers (after removal of trash and larger materials), this would not be attractive due to the number of units required. For example, ten scalpers 48 by 12 feet in size would be required to handle the flow from an 18-inch dredge. Direct processing in scalpers would be feasible only in the case of small-capacity dredges. In other cases, methods for separating and concentrating the material must be used.

<u>Separation Concepts</u>

A number of methods of separating and concentrating the sand and gravel materials were investigated. The prime candidates are considered to be the following:

Separation basins with dragline removal

Separation basins with secondary dredge removal

Hydraulic thickeners

Secondary dredge removal from conventional containment basins

With all of these concepts, sedimentation is used to separate the material which is concentrated on the bottom. In the first three concepts, this is accomplished by limiting the settling basin area to selectively remove the sand-sized materials and to allow the fine-grained materials to carry over with the overflow. In the latter concept, this is accomplished by different settling trajectories of the heavier particles. The basic differences in the four concepts are the methods used to remove the concentrated materials.

<u>Separation Basins</u>

The size of the basin required to selectively remove sand and gravel materials can be determined using the analytical methods described previously. The settling velocity of a 74-micron particle (sg 2.65) is 0.016 feet per second. The required areas based on the flow rates used in Table 1 are:

Dredge Size (inches)	Basin Area (ft^2)
8	250
12	670
18	1,850
24	3,660
30	6,700
36	10,000

As indicated, the size of the separation basins are relatively small. For this reason, the basin would fill rapidly with materials having a large sand and gravel fraction. Using the production rates shown in Table 1, the fill rates would be as follows:

Material	Fill Rate Feet/Hour
Class I	4.4
Class II	2.4
Class III	0.5

The same relationships exist for all the dredge sizes and indicate the retained material must be removed concurrently with the dredging operations. Alternatively, multiple basins could be used and the concentrated materials removed for processing later.

In the case of the smaller dredges, dragline removal of the material could be used since the size of the basins will be within reach. The removed material would be placed into hoppers and would be conveyed either by belts or as a concentrated slurry to the scalpers for processing.

For the larger basins, the alternatives are to use long narrow basins for dragline removal or to use secondary dredges capable of removing and transporting the materials as a concentrated slurry. The secondary dredges could be bucket-type with slurry transport or the horizontal auger cutterhead-type such as manufactured by National Car Rental Systems, Inc., shown on Figure 6, called the MUD CAT.

Figure 7 illustrates the relationship between the primary dredge-size and pumping rate and the required flow rates and solids concentrations of the secondary dredge to maintain concurrent removal.

Hydraulic Thickeners

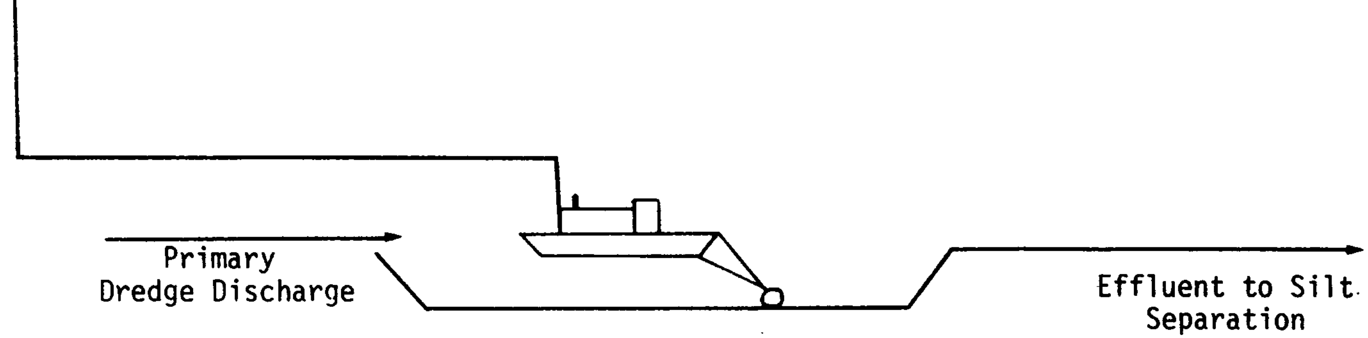
An alternative to secondary dredge removal would be the use of hydraulic thickeners shown conceptually on Figure 8. The required diameter

Fig. 6. MUD CAT Dredge

To Benefaction Equipment

Required Flow Rate For A Secondary Dredge (gpm)

Primary Dredge Size	30% Solids I	30% Solids II	30% Solids III	40% Solids I	40% Solids II	40% Solids III
12	1,570	870	173	1,080	600	120
18	4,480	2,460	490	3,100	1,700	340
24	10,030	5,560	1,110	6,920	3,950	770
30	13,990	9,200	1,800	9,600	6,230	1,240



Primary Dredge Size (in)	Primary Dredge Flow Rate (gpm)	Quantities of Sand and Gravel Material @ 15% Solids (lb/min) Class I	Class II	Class III
12	4,000	4,800	2,670	530
18	11,000	13,700	7,540	1,500
24	22,000	30,700	17,000	3,400
30	40,000	42,800	27,600	5,500

Figure 7. Sand and Gravel Handling Secondary Dredges

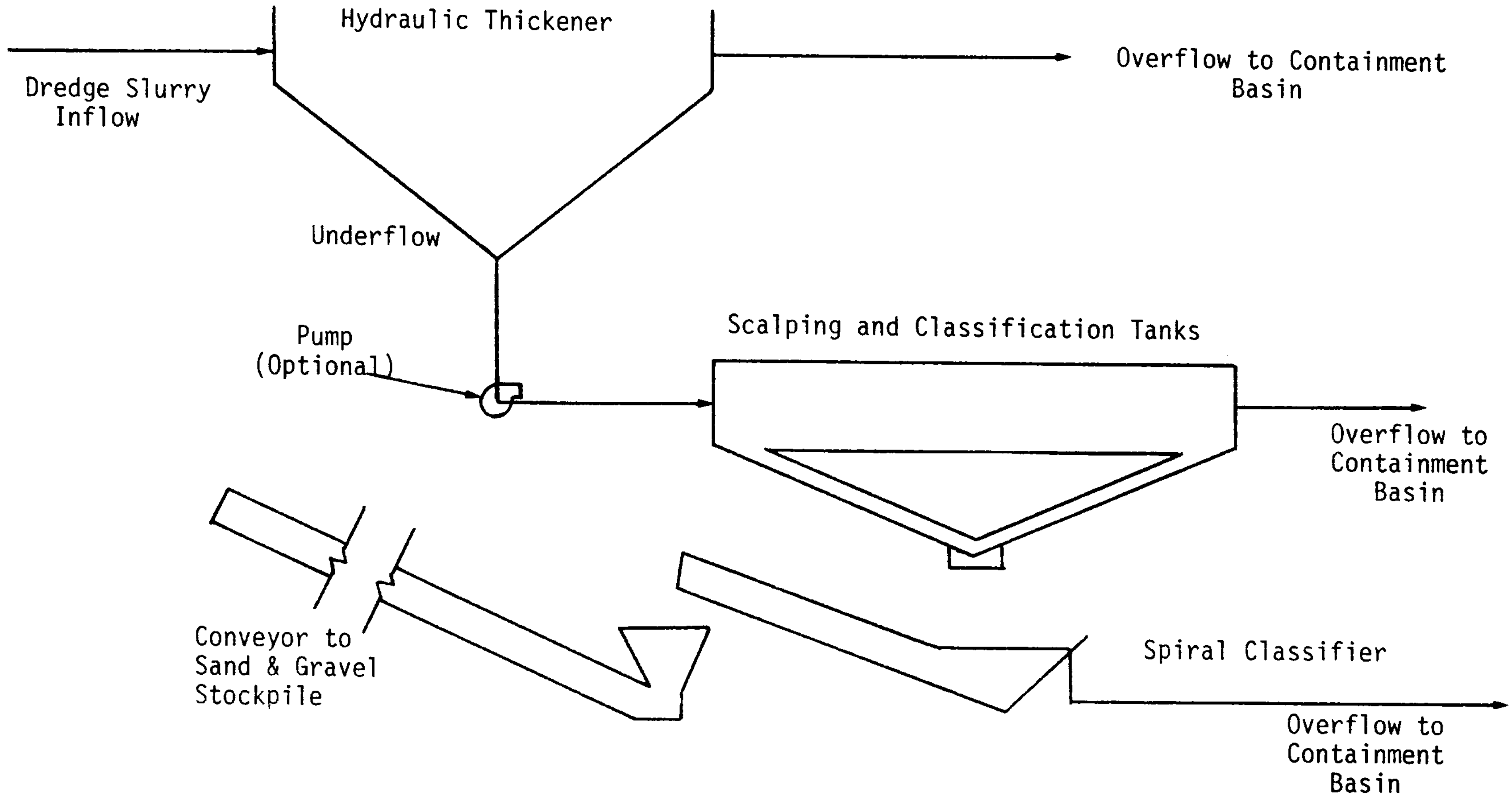
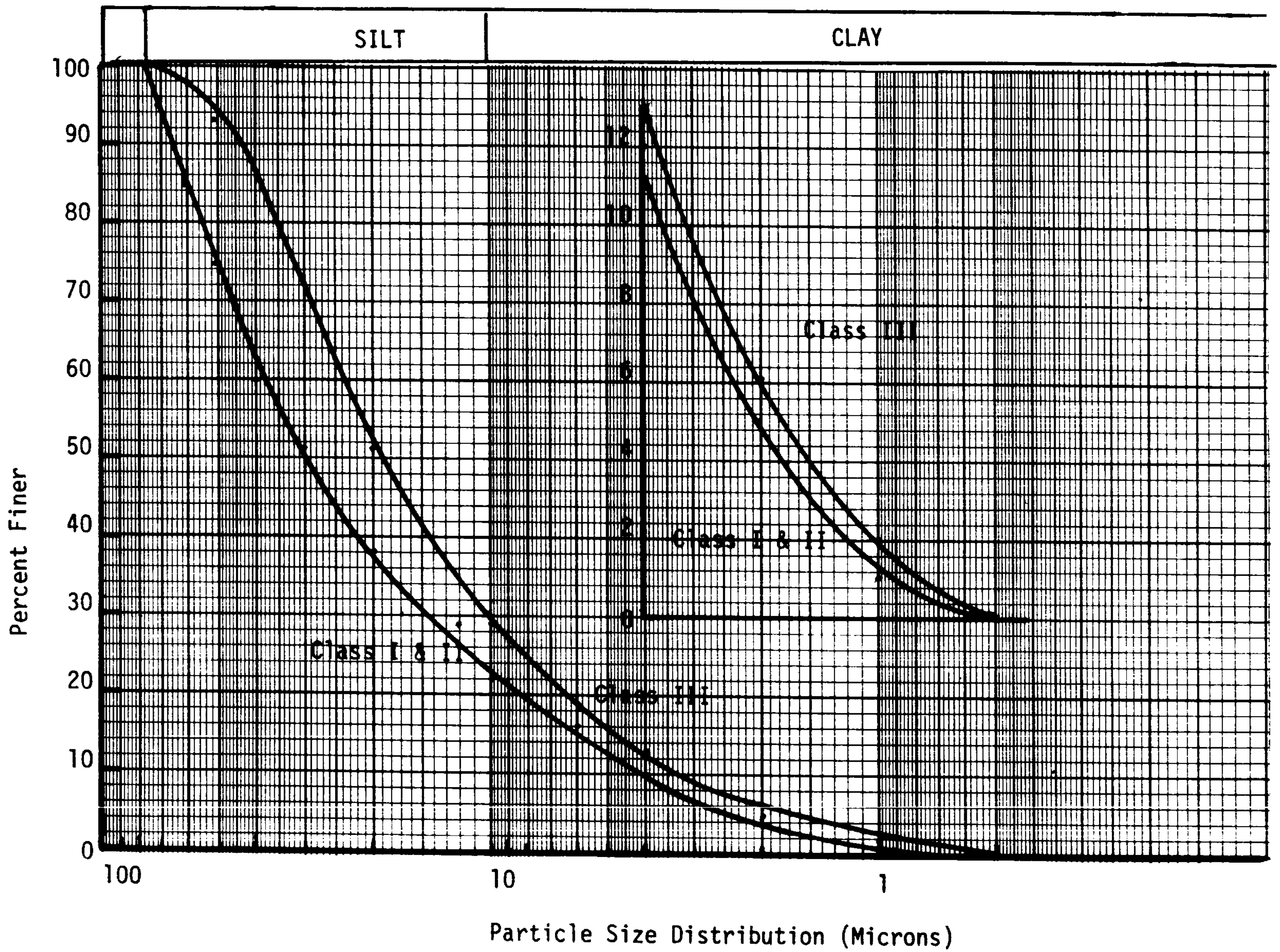


Figure 8. Hydraulic Thickening Concept



PARTICLE DISTRIBUTIONS AFTER SAND AND GRAVEL SEPARATION

Figure 9

for the thickeners and the underflow slurry concentrations are shown on Table 2. This table assumes an underflow concentration of 35 percent solids by weight. Higher concentrations would reduce the underflow.

Number of Scalpers

The number of scalpers that would be required with both secondary dredge removal and hydraulic thickeners would be essentially the same since the slurry would have about the same concentration of solids. The benefits of using these techniques for thickening prior to sand processing are shown in Table 3. As indicated, the concentration of the sand and gravel prior to processing materially reduces the flow rates shown on Table 1 and the processing equipment required.

Table 3. Scalper Requirements
(with and without solids concentration)

Dredge Size (in)	Without Concentration	Number of Scalpers Required* Without Concentration		
		Class I	Class II	Class III
8	1 @ 40'x10'	1 @ 20'x8'	**	**
12	2	1 @ 36'x10'	1 @ 20x8'	**
18	5	2	1	**
24	10	4	2	1 @ 28'x8'
30	19	6	4	1 @ 40'x10'
36	28	9	5	1

* Based on 48 x 12 foot scalpers unless otherwise indicated
** Would use 20 x 8 foot scalpers with part-time operation

Conventional Basins with Secondary Dredges

The sand and gravel fraction of the dredged material will drop to the bottom in the vicinity of dredge inlet. The particle trajectory will be determined by its settling velocity and the current induced by the incoming slurry. Particles in the sand range and above will fall out within a few hundred feet. By controlled positioning of the dredge slurry discharge, the sand and gravel fraction can be selectively placed within conventional containment basins with nominal contamination by fine-grained materials.

The concept of secondary dredge removal from conventional containment basins is similar to concurrent removal using secondary dredges. The difference being that removal and reclamation operations can be conducted at lower rates and thereby use less equipment than concurrent processing.

Table 2. Hydraulic Thickener Diameters and Underflow Rates

Dredge Size (in)	Thickener Diameter (ft)	Underflow Rates (gpm @ 35% wt)		
		Class I	Class II	Class III
8	18	500	280	55
12	29	1,290	715	150
18	48	3,620	2,030	400
24	68	8,250	4,600	900
30	92	13,300	7,400	1,500
36	112	19,900	11,000	2,200

An example of this type of operation would be a single dredge of the MUD CAT type pumping materials to a single 48 by 12 foot scalper. At pumping rates of 2000 gpm and solids concentrations of 30 percent, about 150 TPH of sand could be reclaimed and processed per hour assuming 20 percent contamination of the inplace material by finer solids.

SILT SEPARATION AND HANDLING

With the removal of sand and gravel fraction using separation basins or thickeners, the particle distribution of the remaining slurry would be typically as shown in Figure 9. In addition, the solids concentration of the slurry would be reduced. Based on an incoming slurry concentration of 15 percent prior to sand and gravel separation and the assumed material class shown in Figure 1, the concentration of the overflow from sand separation would be as follows:

Material	Solid Concentration (wt %)
Class I	1.5
Class II	7.5
Class III	13.5

This material could be separated and stored in conventional containment basins with a significant reduction in volume in the case of Class I and II material. However, further processing would be required to effect significant reductions in the case of Class III materials.

Separation and Handling Concepts

Of the concepts studied, the following were considered suitable for application:

- Silt separation and dewatering basins
- Silt separation basins with secondary dredge removal
- Secondary dredge removal to on-site dewatering basins
- Secondary dredge removal with pumping to off-site dewatering basins
- Thickeners with pumping to off-site dewatering basins
- Hydraulic placement directly as fill

Selective Removal of Silt

The dewatering characteristics of silt and clay mixtures are generally very slow if a significant fraction of clay is present. If there are no clays present, silt with minimum particle sizes in the 10 to 20 micron range has relatively good dewatering characteristics and can potentially be used as fill materials. For these reasons, it is desirable to selectively remove the silt fraction and minimize the clay content. As previously illustrated in Figure 4, the silt fraction represents some 70 to 80 percent of the solids remaining after sand and gravel removal. The reclamation of silt for off-site utilization can also significantly reduce the storage requirements if conventional containment basins are used for the remaining materials.

Because the silt and clay materials will be in a relatively dilute slurry, the most practical method of separating the silt will be by sedimentation. The settling velocities of 10 and 20 micron silt particles are 2.92×10^{-4} and 1.17×10^{-3} ft/sec., respectively. On a theoretical basis, the size of the basins that would be required to selectively remove silt are shown in Table 4, based on the flow rates shown in Table 1. It is noted that these areas are based on continuous flow. With intermittent operation, finer-grained particles will settle out. As discussed later, other clay-sized particles may also be retained because of particle agglomeration.

Table 4. Area Requirement Silt-Removal Basins

Dredge Size (in)	Basin Area ft^2 & (acres) 10 micron	20 micron
8	11,400 (0.26)	2,860 (0.07)
12	30,400 (0.7)	7,650 (0.18)
18	84,000 (1.9)	21,000 (0.43)
24	167,000 (3.8)	42,000 (1.0)
30	304,000 (7.0)	76,000 (1.8)
36	458,000 (10.5)	115,000 (2.6)

Removal from Silt Basins

The size of the silt separation basin generally precludes the use of draglines. Two alternatives exist for the handling of the material. The first is to use multiple basins designed to also function as de-watering basins. After filling and dewatering, the dikes would be breached and the material removed by front loaders or shovels and loaded into trucks for off-site transport.

The second alternative would be the use of secondary dredges as previously discussed for sand separation basins. In this application, the material would be pumped, either as a slurry to dewatering basins located onsite, or, with the use of booster pumps, to locations several miles off-site. Figure 10 indicates the horsepower required for slurry transport to off-site locations.

Use of Thickeners

Thickeners might also be used to separate silt. However, with the larger-sized primary dredges it would not be possible to remove particles in the 10 to 20 micron range. The largest thickeners built are in the range of 250 feet in diameter due to limitations in the construction of the rotating scraper. Where greater settling areas are required, multiple thickeners are used. Table 5 shows the applications of thickeners within the 250-foot diameter limit.

Table 5. Thickener Diameter for Silt Separation

Dredge Size (in)	Particle Size Removed (microns) 10	20	25	30
	Thickener Diameters (feet)			
8	121	60		
12	197	98		
18		166		
24		230	184	
30			248	207
36				228

Thickeners would eliminate the need for secondary dredges and would provide the capability of handling large quantities of silt on

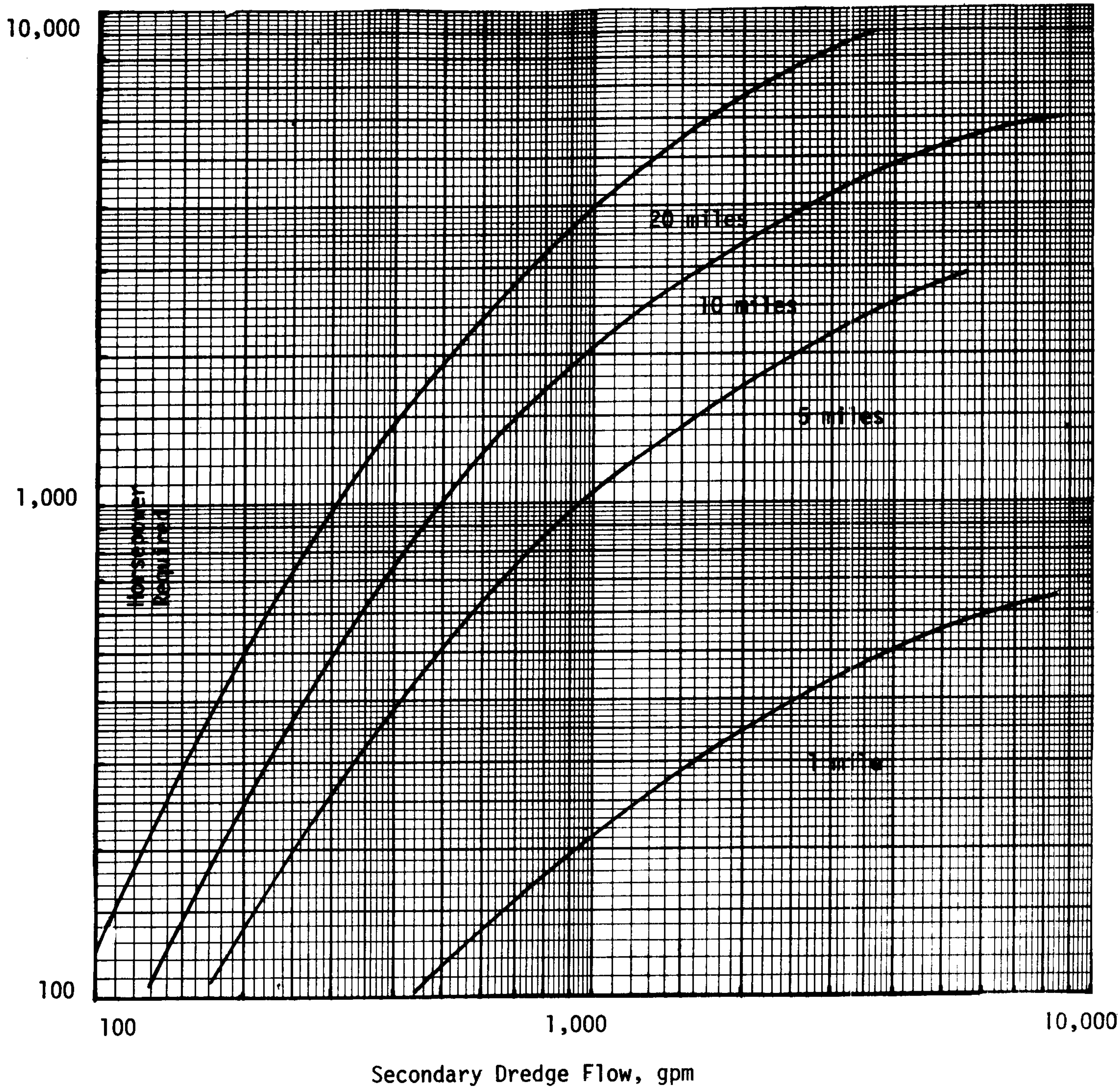


Figure 10. Estimated Pumping Requirements for Off-Site Disposal (40 percent solids concentration)

a continuous basis. Many maintenance dredging operations will involve materials with the Class II to Class III range as shown on Figure 1. As indicated on Table 1, the quantities of silt to be handled will be in the 400 to 1650 ton-per-hour range, for dredges in the 24 to 36 inch range. Assuming solids concentrations from the thickeners in the range of 35 percent, the sizes of the slurry pipelines required to move these materials are shown in Table 6.

Table 6. Slurry Pipeline Requirements

Dredge Size (in)	Material Class	Total Solids (TPH)	Slurry Flow Rate (gpm)	Pipe Diameter (in)
28	II	408	3,840	12
	III	683	6,400	16
30	II	660	6,150	16
	III	1050	9,800	18
36	II	998	9,300	18
	III	1650	15,400	20

Hydraulic Placement of Fill

In certain cases, it may be possible to place the silt slurry directly as fill material. This can be done where the area to be filled can be contained with dikes. In some cases placement of a layer of sand and gravel with drains or well-points may facilitate the silt-dewatering process. Dewatering may also be facilitated by working of the material.

EVALUATION OF SILT SEPARATION BASINS

A field evaluation was made of two silt-removal basins used in conjunction with a small-scale dredging operation. The material being dredged contained very little material of sand-sizes and above, and the slurry was characteristic of what one would expect following sand and gravel separation. The basins were roughly 150 by 170 feet with depths of 12 feet, and used temporary earthen dike construction. Figure 11 is a photograph of one of the basins.

The sampling program consisted of taking inlet and effluent samples and bottom samples of the material being deposited. Particle distributions were determined using the hydrometer method for the inlet and bottom samples. Hydrometer tests were conducted with and without dispersants. Samples were also collected to determine the moisture content of the dewatered

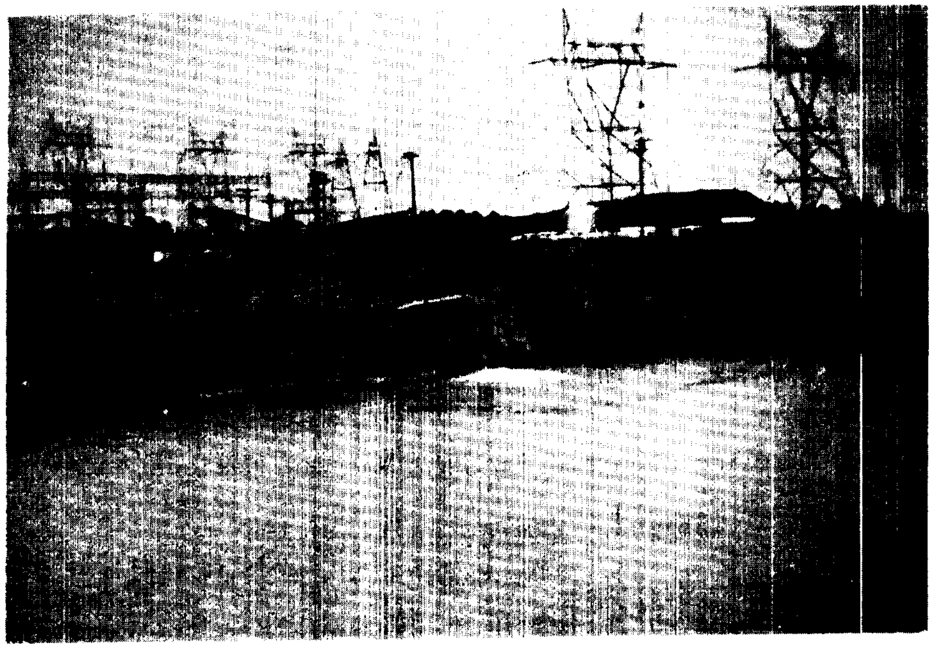


Figure 11. Silt Removal Basin

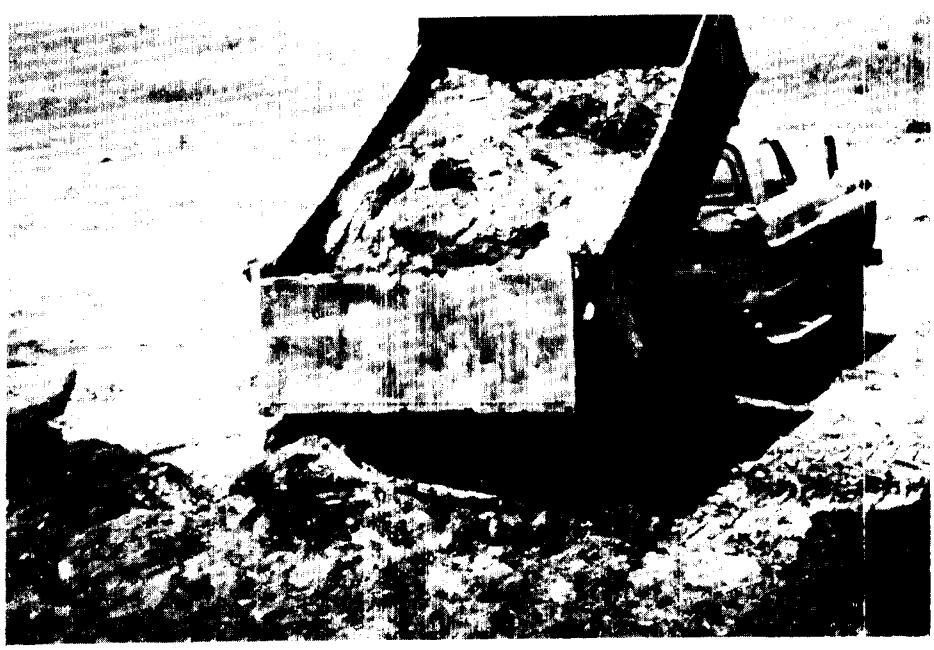


Figure 12. Truck Handling Removed Silt

material prior to removal.

Overall Removals

The results of these tests are shown on Table 7. The ideal predicted removal was calculated based on the flow rates, area of the basin, the dispersed particle distributions and idealized theory. As indicated, the actual removals were considerably higher. This is attributed to the agglomeration of fine-grained particles which had higher settling velocities than would be indicated by the particle distribution obtained by dispersed hydrometer analysis. In addition, the dredging operation was intermittent and additional settling occurred during quiescent periods.

Table 7. Removal Efficiencies

Influent Concentration (mg/l)	Effluent Concentration (mg/l)	Actual Removal (%)	Potential Removal (%)
88,860	1,172	98.8	60
46,160	639	98.7	72
121,710	1,420	99.0	57

Bottom Samples

Based on idealized theory and continuous operation, the minimum particle size that would be retained by the basin was eight to nine microns. At the upper end of the basin, 0 to 30 percent of the particles were smaller than predicted. At the lower end, 30 to 70 percent of the particles were smaller than predicted. These differences are attributed to the intermittent operation and the higher settling rates of agglomerated particles.

Dewatering Characteristics

After the basin was filled, the surface water was decanted and the material was allowed to dewater for two weeks. The water content of the material at the outlet end of the basin ranged from 35.5 to 44.2 percent. This area of the basin contained a large fraction of material with particles less than 10 microns in size. The material was capable of supporting the weight of a man but would flow when subjected to repeated stress.

The material was removed by breaching the dikes and was loaded using front loaders and shovels into trucks. Figure 12 shows a truck dumping the material. The material was spread in layers up to one foot thick;

dewatering continued, and the material was capable of supporting mechanized equipment in about a week.

Test Conclusions

The following was concluded based on the results of these tests:

- Removal efficiencies were greater than predicted by theory.
- Non-dispersed hydrometer samples gave a better representation of settling characteristics than dispersed samples.
- Silt-separation basins should be made smaller than predicted by theory in order to reduce contamination by clay materials.
- Even though the material contained large fractions of clay-sized material, the dewatering characteristics were good, indicating that agglomerated clay particles did not inhibit dewatering.
- The silt separation concept is considered feasible to reduce the area and storage requirements of containment facilities.

HANDLING OF CLAYS

With the removal of the gravel, sand, and silt, the quantities of solids to be handled will have been reduced significantly. The quantities of solids will probably be even less than 2, 10 and 23 percent for Class I, II, and III materials. As indicated in the silt-separation discussion, a significant fraction of the clay-sized materials may be removed as agglomerated particles with the silt. However, the clay-sized materials represent a more demanding problem than the larger materials. They are difficult to separate, pump, dewater and handle.

Water Quality Considerations

With both conventional containment facilities and with dredged material separation facilities, a significant portion of the clay materials must be removed because of water quality considerations. Figure 13 shows the particle distributions expected after removal of gravel, sand and silt to 10 microns. The particle distributions of the clay-size materials will be essentially the same even though the original slurry contained varying amounts of larger particles. The concentrations of the slurries at this point will be significantly different depending upon the amount of solids separated in the preceding processes.

Table 8 shows the removals required to meet various water quality goals and the size particles which must be removed. These are calculated using the procedures described in the introduction and the assumed distributions shown on Figures 1 and 13.

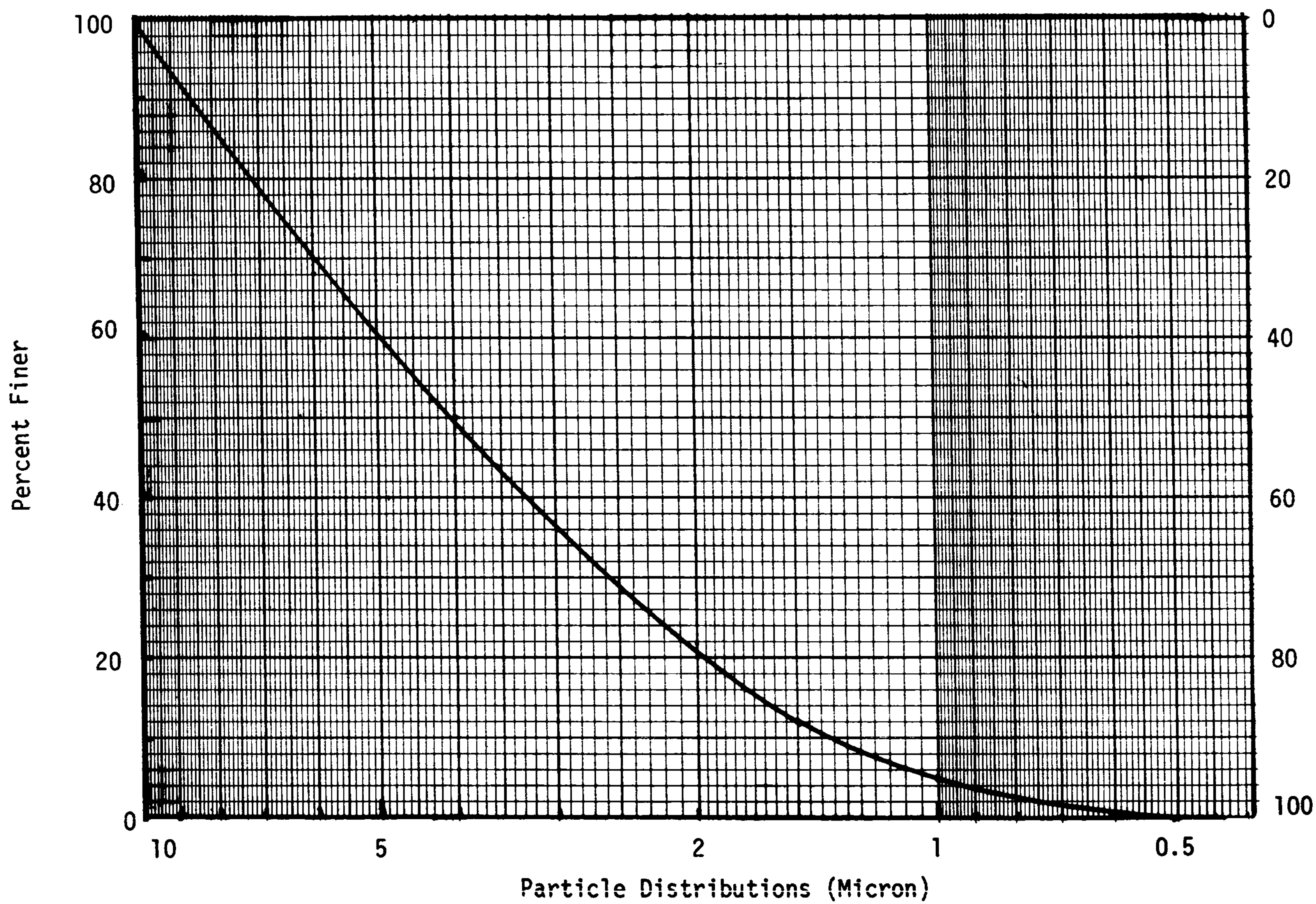


Figure 13. Clay Particle Distributions After
Removal of Silt to 10 Microns

Table 8. Required Removals to Meet Water Quality Goals

Material Class	With Gravel Sand & Silt Removal	Slurry Concentration (%)	Water Quality Goal (gm/l) 8	6	4	2	1
			Percent Removal (Particle Size to be Removed - Microns)				
I	No	15	95.4 (27)	96.6 (18)	97.7 (12)	98.9 (6)	99.4 (3)
	Yes	0.3	*	*	*	33.5 (5.5)	66.8 (2.4)
II	No	15	95.4 (8)	96.6 (4)	97.7 (3)	98.9 (1.5)	99.4 (0.8)
	Yes	1.5	47.0 (4.3)	60.4 (3.3)	73.7 (2.3)	86.8 (1.6)	93.4 (1.1)
III	No	15	95.4 (2.3)	96.6 (1.7)	97.7 (1.2)	98.9 (0.8)	99.4 (0.7)
	Yes	5.2	85.3 (.17)	89.1 (1.5)	92.7 (1.2)	96.3 (0.88)	98.2 (0.75)

* Further removal not required

Although the percentage of material which must be removed is reduced by the removal of gravel, sand and silt, the size particle which must be removed to meet a given water quality remains essentially the same. This illustrates how the clay-size materials dictate the size of containment facilities.

Containment Basins

The area of the basins required to retain the particle sizes indicated can be estimated using the relationships previously discussed. The required areas are shown in Table 9.

These estimates are considered to be conservative because of the agglomeration of particles and higher removal efficiencies observed in silt separation basins and with conventional containment facilities. However, they do illustrate the large land-usage requirements that would be involved in meeting more stringent water quality requirements than presently exist.

Storage Considerations

A clay-containment basin sized to meet an eight-gram-per-liter water quality requirement will provide a large amount of storage for the clay fraction alone. This can be illustrated by an example. From Table 1, the processing rate for clay with an 18-inch dredge and Class III material is 105 TPH. Allowing for bulking, this equates to about 3000 cubic feet of solids per hour. The 58-acre basin required to attain an eight-gram-per-liter effluent would fill only to a depth of one foot in 840 hours of dredging. If the gravel, sand and silt had not been separated, the fill rate would be about four times this amount. For materials having lesser clay fractions and for basins having larger areas to meet more stringent water quality goals, the fill rates would be significantly less.

Alternative Concepts

The use of conventional containment basins to retain the clay fraction still represents a significant land use in areas of extensive maintenance dredging. It would be desirable to reduce the land area required to meet water-quality goals and the long-term storage requirements. The concepts by which this can be done include:

Table 9. Required Containment Basin Areas

Dredge Size (in)	Material Class	Water Quality (gm/l) 8	6	4	2	1
		Area (Acres)				
12	I	*	*	*	20	14
	II	5	8	16	33	85
	III	21	27	59	132	174
18	I	*	*	*	6	37
	II	13	21	44	91	232
	III	58	75	161	363	477
30	I	*	*	*	23	134
	II	46	78	159	328	840
	III	210	269	584	1313	1726

- Use of coagulants to enhance settling
- Use of secondary dredges to transport concentrated clay slurries to off-site disposal areas

Use of Coagulants

In most of the uses of coagulants in dredging operations, the coagulant has been added to the slurry. In certain cases, significant improvement in return water quality has been attained, however, the dosages required have been relatively high.

The use of separation systems to remove the materials other than clay can potentially improve the use of coagulants. When mixed with slurries containing heavier particles, a significant amount of the coagulant can be lost by settling with the larger particles. When added to slurries composed almost entirely of clay particles, the coagulant is given a better opportunity to contact the particles. This is particularly true to the long-chain synthetic polymers and polyelectrolytes. Limited data on the use of polyelectrolytes would indicate that containment areas can be reduced by factors of four to ten with the use of polyelectrolytes.

Use of Secondary Dredges

The use of coagulants to reduce the area of clay containment basins will also reduce the storage capacity and it will be necessary to remove the material. This can be accomplished using the secondary dredge removal concept discussed for both sand and silt processing. Because the clay retention basins will be still relatively large, they will generally be adequate to temporarily store the clay materials during most dredging operations. Where this is the case, the secondary dredging operation can be used to restore the containment facility between dredging operations.

Most clay materials are thixotropic and exhibit non-Newtonian behavior when pumped as a slurry. For this reason, the pumping power required for off-site transport of slurries can be significantly higher than for other materials. However, since the quantities of material will be a fraction of the total solids, this will generally not be a significant factor in the overall handling system.

Use of Temporary Basins

The ideal settling basin is a large-area shallow basin from the standpoint of solids retention. Shallow basins are generally not used because of storage considerations. With the use of secondary dredge removal, the use of shallow basins is feasible. These small dredges operate with less than two feet of water and basins with depths of four to five feet can be considered. With this depth, simple earthen dikes can be used. Because the retained material will be removed, the construction can be temporary.

ECONOMIC ASPECTS

The cost of applying the concepts described above will be dependent upon the type and quantity of material to be dredged and other local conditions. For this reason, only generalized estimates of the capital and operating costs have been made.

Sand and Gravel Processing

Assuming a 24-inch dredge and Class II material, the equipment and installation costs of sand separation basins with secondary dredges and hydraulic thickeners were estimated to be in the range of $260,000. These estimates included the cost of the scalpers, spiral classifiers, and conveyors needed to process the material into stockpiles. This system would be capable of processing in excess of one million tons of material per year. The annual operating costs were estimated to be $300,000 to $400,000 per year, including allowances for capital recovery, insurance, taxes, etc.

The cost associated with the processed material was estimated to be in the 27 to 35 cents per ton range which compares to a unit value of $1.18 for all sand and gravel sold in the U.S. in 1971. The production costs would be subject to fluctuation depending upon the sand and gravel fraction present.

Silt and Clay Handling

For the same conditions the costs were estimated for removing silt using secondary dredges and transporting the material as a slurry to a location five miles off-site. The estimates included capital recovery, insurance, taxes, in addition to operating and maintenance costs. Based on moving 740,000 cubic yards annually, the unit cost would be in the range of 65 cents per cubic yard of solids. The comparable cost of truck transport would be about one dollar per yard not including cost of dewatering the material on-site and loading it into trucks.

The cost of clay removal would be higher due to the higher pumping costs for a given quantity of material. For this example, clays constituted only ten percent of the material dredged. On this basis, the cost of handling the clay would add about ten cents to the cost of dredging. However, this cost could be more than offset by the utilization of the other materials.

Other Benefits

The greatest economic benefit to be derived through the use of dredged material separation systems will be the savings that will result from not having to acquire land and construct new facilities. In many cases, the cost of the separation equipment can be fully recovered by being able to use less land. The operating and maintenance costs of the separation equipment can be more than offset by the use of the recovered products.

New containment facilities will also involve a number of indirect costs. These include siting studies, land acquisition, facility planning, environmental assessments, public hearings, etc. These indirect costs alone could represent a significant portion of the cost of converting existing containment facilities to separation and reclamation facilities.

THE RESPONSES OF SOME ESTUARINE ORGANISMS TO SUSPENDED SOLIDS

By

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INTRODUCTION

The necessity to develop further our understanding of animal-sediment relationships has become abundantly obvious in the past five years. Some of the reasons for this have already been outlined in this seminar by Capt. Meccia of the Army Corps of Engineers. Proposals for increased engineering activities in freshwaters, estuaries, the coastal zone and the outer continental shelf are now being acted upon in government. Each of these proposed activities raises serious environmental questions. Paramount among these is: What is man's ability to effect a meaningful compromise between resource exploitation and ecosystem conservation?

We find ourselves thrust into the science of bio-engineering, for want of a better term, within which our ability to maintain or cut new channels, to mine sand and gravel from the sea-bed and our choice of waste-disposal sites, oil drilling sites and other activities will be regulated, criticized and, only in some cases, passed upon.

As alluded to in the course of this seminar, we possess the hardware to execute virtually all the dredging activity contemplated now, and in the future. The hardware, however, is only half the problem. To preserve the quality of life as we know it today we must now examine how, when and where we can operate that hardware because the aquatic environment, even the vastness of the open sea, represents a finite resource. We must proceed carefully in order to avoid unforeseen tragic consequences.

In this presentation I outline, in very general terms, some of the results of a research project undertaken to assess the impact of suspended solids on aquatic organisms, in particular estuarine organisma common to the bays and rivers of the Atlantic Coast of North America. We also propose some guidelines based upon our research results which should be applied in approaching dredging and dredge-related activities from the point of view of environmental impact. The results presented here are intended to document some of the effects observed as the result of exposing organisms to suspended solids. It is not intended that these data be taken to indicate that all organisms do or would respond in a similar manner; however, effects experienced by even a few species in an ecosystem may eventually affect the structure and function of that system. Thus, the fact that effects are inducible is, of itself, important to consider when dredging operations are planned or undertaken.

Studies funded by the U.S. Army Engineers Coastal Engineering Research Center were undertaken in 1970 at the University of Maryland Natural Resources Institute, to investigate the effects of suspended and deposited sediments on estuarine phytoplankton, zooplankton and fish. A scope of work was established at the outset which established the following objectives:

1. To determine the effects of particle size and particle concentration on the survival of selected estuarine organisms;
2. To assess the effects of particles on the function of estuarine organisms, i.e. sublethal effects;
3. To determine, at least in part, the factors and areas of concern which may play a significant role in determining the outcome of a direct confrontation between dredging and natural aquatic systems; e.g. impact on water quality, B.O.D., nutrients, adsorbed heavy metals, adsorbed pesticides, and the like.

We gratefully acknowledge the following persons and institutions for their assistance and support during the course of the studies: Drs. L. Eugene Cronin, Joseph Mihursky and Raymond P. Morgan, and Messrs. D.A. Neumann, R. Prince, A. Daley, K.Wood and G. Cox, of the University of Maryland; Drs. R. Biggs, K.Price, and the staff of the University of Delaware Bayside Laboratory; Dr. J. Lawler, and Ms. C. Goroshko of Lawler, Matusky and Skelly Engineers, Tappan, N.Y.

MATERIALS AND METHODS

Studies were carried out in the laboratory using organisms from two estuarine areas, the Patuxent River estuary, Maryland and the Lower Delaware Bay. Studies in Maryland were carried out at the University of Maryland Natural Resources Institute Laboratory at Hallowing Point. Studies in

Delaware were carried out at the Bayside Laboratory of the College of Marine Studies, University of Delaware. Our studies with wild animals concentrated upon fishes (Table 1) although some work was carried out using some of the more common macro-invertebrate fauna found on the Atlantic Coast, such as the blue crab (Callinectes sapidus) and the American oyster (Crassostrea virginica).

Studies of the planktonic organisms were carried out using cultures of "typical" estuarine forms; several phytoplankters were obtained from the cultures of the Woods Hole Oceanographic Institute (Table 2). The estuarine copepods Acartia tonsa and Eurytemora affinis were obtained from the cultures of Dr. Donald Heinle of the University of Maryland.

Species selection was such that the organisms tested represented forms common to upper estuarine zones (oligohaline zones) where low and moderate salinities prevail, such as in the Patuxent River estuary, as well as lower estuarine zones (mesohaline zones) where moderate to high salinities prevail, such as in the lower Delaware Bay. Extensive coverage of the various estuarine zones was required based upon the objective of the studies and upon the physical aspects of fine sediment distribution in estuaries having typical salinity gradients.

Materials and methods employed in the various aspects of this study may be gleaned from the final report (Sherk, O'Connor, Neumann, Prince and Wood, 1974) to be available soon from N.T.I.S. In summary, the effects of suspended particles on primary productivity were estimated using the ^{14}C method (Strickland and Parsons, 1972). The effect of particles on filtration (=feeding) rates of zooplankton was determined in culture by feeding ^{14}C labeled phytoplankton to two species of copepods, Acartia tonsa and Eurytemora affinis at various concentrations of suspended solids. The lethal and sublethal effects of suspended particles on fishes were estimated by acute and chronic bioassay experiments (Doudoroff et al., 1951).

A variety of materials was used in an effort to isolate the effects on the test organisms of particle size, mineral composition of the solids, particle concentration and sorbed materials (Table 3). The demands of experimental design and time prevented replication of each test with all the materials tested; most tests used fuller's earth, hydrite-10 (Georgia Kaolin Co.) or silicon dioxide (sand), and were repeated using concentrations of solids made up on resuspended natural muds. Hydrite-10 and fuller's earth conformed most closely in particle size distribution to the natural Patuxent River muds used

Table 1. Species used in evaluating the effects of suspended mineral solids on estuarine fishes.

Species	Common name[1]	Location of[2] capture	Capture[3] method
Brevoortia tyrannus	Menhaden	Del.	H.S.
Anchoa mitchilli	Bay anchovy	Del.	H.S.
Fundulus majalis	Striped Killfish	P.R.	H.S.
F. heteroclitus	Mummichog	P.R.	H.S.
Rissola marginata	Cusk eel	Del.	H.S.
Menidia menidia	Atlantic silverside	Del.	H.S.
Morone saxatilis	Striped bass	P.R.	O.T.
M. americana	White perch	P.R.	O.T.
Leiostomus xanthurus	Spot	P.R.	O.T.
Micropogon undulatus	Croaker	P.R.	O.T.
Cynoscion regalis	Weakfish	Del.	H.S.
Trinectes maculatus	Hogchoker	P.R.	O.T.
Pomatomus saltatrix	Bluefish	Del.	H.S.
Opsanus tau	Oyster toadfish	P.R.	O.T.

1. Amer. Fish. Soc. Spec. Pub. no. 2

2. Del. = Bayside Laboratory, University of Delaware, Lewes, Delaware
P.R. = Patuxent River Estuary, Maryland

3. H.S. = 50' beah seine
O.T. = 20' Otter Trawl pulled at 3 k for 3-5 min.

Table 2. Algal forms employed to study the effects of suspended sediments on estuarine plankton.

Name	WHOI Clone Symbol
Monochrysis lutheri	Mono
Nannochloris sp.	GSB Nanno
Stichococcus sp.	GSB Sticho
Chlorella sp.	0-10

in the experiments.

RESULTS AND DISCUSSION

The results showed clearly that suspended particulate matter may affect the productivity of natural waters and may, under certain circumstances, pose a threat to the survival of fishes and fisheries.

The results presented in overview form in this report show "effect" levels of suspended particles to be rather much greater than concentrations which could be found in natural circumstances or in the vicinity of dredging activities (see e.g. Masch and Espey, 1967). Justification of these studies however, is founded in the fact that the organisms used in these studies are, of themselves insensitive to the effects of handling and manipulation. One cannot work effectively with an organism which is highly sensitive to handling alone, and our ability to document the potential for impact due to suspended solids on relatively hardy organisms and life history stages suggests that less hardy forms would be proportionately less able to tolerate the same, or less severe, concentrations.

PRIMARY PRODUCTIVITY

Carbon assimilation in phytoplankton was tested in relation to a gradient of concentrations of sand (SiO_2) ranging from 0.2 to 2.0 grams per liter ($g\ l^{-1}$) and was found to decrease in proportion to the concentration of material

Table 3. Materials employed in experimental determinations of the effects of suspended solids on estuarine organisms.

Material	Median Particle size (Microns)	% less than 2.0μ	Mineral Composition	Source
Fuller's earth	0.50	82.0	Aluminum-magnesium Silicate (montmorillonite/attapulgite)	Fisher Chemical
sand	17.00	6.0	SiO_2	Fisher Chemical
Hydrite-10	0.55	92.0	$Al_2SO_3 \cdot 2SiO_2 \cdot 2H_2O$	Georgia Kaolin
Hydrite-MP	9.50		$Al_2SO_3 \cdot 2SiO_2 \cdot 2H_2O$	Georgia Kaolin
Hydrite-flat-D	4.50	34.0	$Al_2SO_3 \cdot 2SiO_2 \cdot 2H_2O$	Georgia Kaolin
Patuxent River sediments	0.78	72.0	---	

used (see e.g. Figure 1). The shape of the curve of photosynthesis reduction was basically a reflection of light extinction. Primary production was affected by several types of particles in the same manner at sediment concentrations from 0.1 to 1.0 g l^{-1}. Some differences in the shapes of the reduction curves occurred between species dependent upon the motility of the algae tested, species-specific light saturation values and the light attenuation characteristics of the particles used in the test. For all species additional concentrations of particles resulted in a further reduction of carbon fixation. Reductions of up to 80% were observed regularly for phytoplankters exposed to concentrations similar to those which have been observed in the vicinity of dredging operations as well as during estuarine flood conditions. The major factor identified in reduction of productivity was particle size; the larger particles (SiO_2) having less of an impact than the smaller particles. This result may be confounded, however, by the fact that the larger particles absorbed more light than the smaller particles tested. It may also be related to other physical properties of the particles.

Several points should be mentioned with regard to the potential impact of suspended solids on primary productivity: First, the particle concentration and light extinction may act synergistically to affect the potential productivity of natural waters. Second, a differential effect may be produced by suspensions of different materials depending upon the light absorption capacities of the material in suspension. For example, suspended solids which absorb light in the red, far-red portion of the spectrum have the potential to affect production more than those which absorb in the green or blue, since chlorophyll a, the most widespread photosynthetic pigment in the major divisions of algae, absorbs light energy in the red, far-red range.

Third, suspended solids may have no effect on estuarine productivity, but simply favor photosynthesis by shade-tolerant algal species which abound in turbid, estuarine waters.

The effects of dredging and spoil disposal on primary productivity are among the most difficult to study. Further work on this question is required, particularly in the context of continental shelf waters, where plans for engineering activities abound, and where significant reduction of primary production may have disastrous effects.

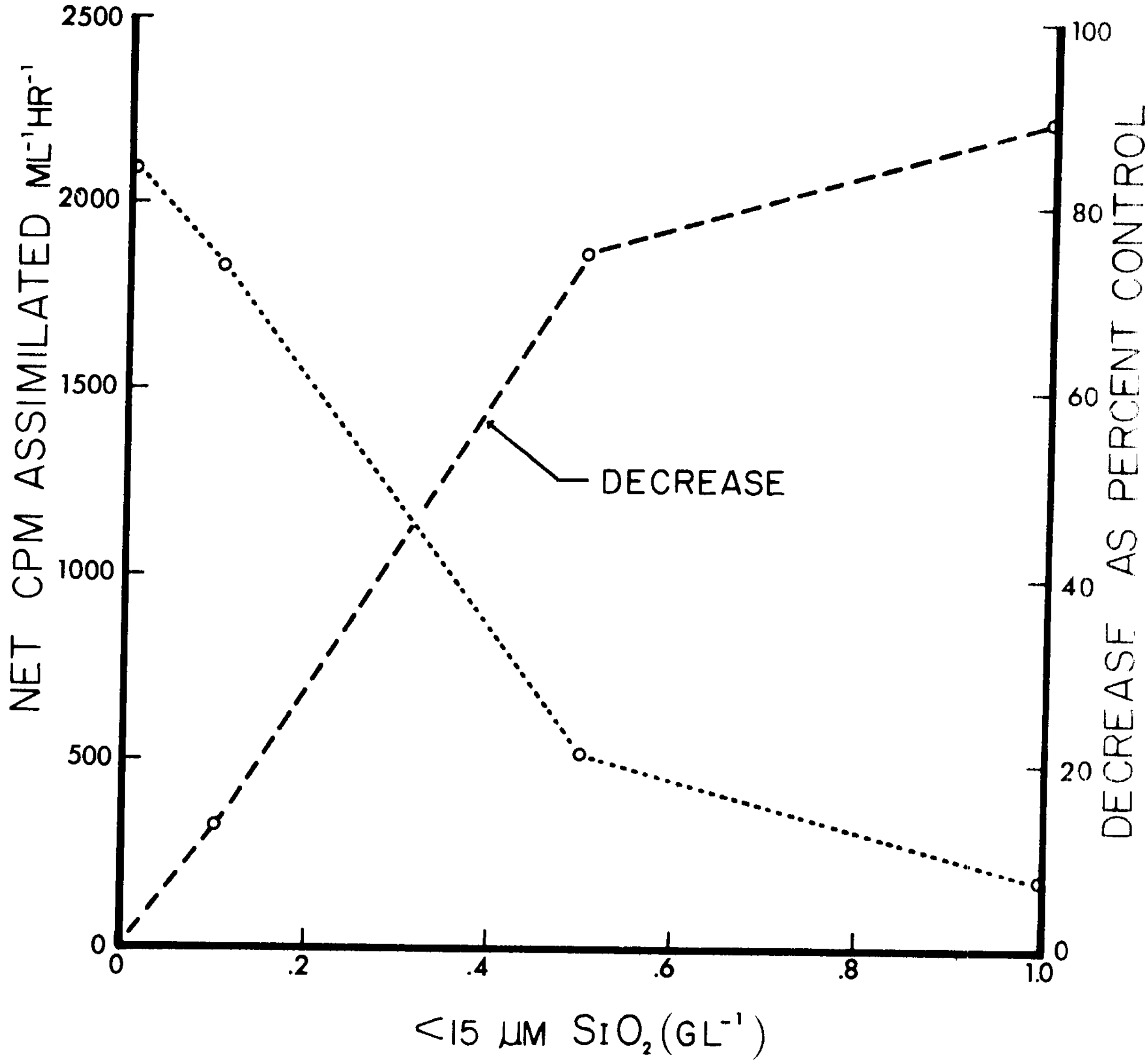


Figure 1. The effect of increasing concentrations of silicon dioxide on carbon assimilation of <u>Monochrysis lutheri</u>.

ZOOPLANKTON

The effect of suspended solids on two filter-feeding copepods was tested successfully. The changes observed in feeding rates for two species of copepods, *Eurytemora affinis* and *Acartia tonsa*, when exposed to suspended particles, differed markedly. For example, filtration of the alga *Monochrysis lutheri* by *A. tonsa* decreased and remained low in 500 mg l^{-1} suspensions of natural silt (Patuxent River sediment). *E. affinis*, however, showed an initial decrease in rate of filtration, followed by an increase which, after three hours, was equal to control values.

The variation in the results observed may be related to the ecology of the organisms concerned. *E. affinis* responded to increased particle concentrations with an increase in the rate of feeding. Similar results have been observed for other filter feeders, namely oysters (*C. virginica*) and the brine shrimp *Artemia salina*. *E. affinis*, a true estuarine form common in the turbid oligohaline and freshwater sectors of estuaries, is apparently stimulated to begin feeding, or to increase its ingestion rate under turbid conditions. The survival value of this strategy is evident for the estuarine habitat; the turbid periods in an estuary (spring through fall) are periods when algal production is at high levels, and when the natural suspended detritus in the estuary is of high organic content. Thus, for the estuarine habitat, turbidity is, in a sense, equatable to available food, and increased feeding will result in optimization of population growth.

A. tonsa, on the other hand, inhabits generally less turbid waters. The uncharacteristic event tested in our experiments, the exposure of *A. tonsa* to high concentrations of suspended particulate matter is probably not a condition experienced frequently in the natural habitat.

These results may be of singular importance in making assessments of dredging and spoiling effects on zooplanktonic organisms in mesohaline and euhaline sectors of estuaries. Increased concentrations of fine particles in environments which are not characteristically turbid, and populated by suspension-feeding organisms which do not respond to increased turbidity with increased rates of filtration, could result in marked decreases in secondary productivity. That is, as turbidity increases, the probability increases that a sediment particle rather than an algal particle will be ingested. Thus, the rate of accumulation of nutrients would decrease, and population characteristics related to nutritional status of the organism would be affected adversely.

FISH

Studies were undertaken to determine lethal and sublethal effects of suspended solids on a variety of fish species. Efforts were made to determine the mechanisms whereby suspended particles exert their effects, as well as the potential adverse ramifications of various events observed in the course of testing; e.g. ingestion of suspended solids, altered activity patterns and the like.

LEVELS OF SENSITIVITY TO SUSPENDED SOLIDS

Studies of the lethal effects of suspended solids were conducted as closed-system bioassays (Doudoroff et al., 1951; Sprague, 1969). Results of the tests were variable depending upon the species used; when placed in the perspective of other studies, such as that of Rogers (1969; see also Saila, Pratt and Polgar, 1972) the LC 50 values determined for estuarine fishes in general may range from a low value of 2.5 g l^{-1}(juvenile menhaden, *Brevoortia tyrannus*) to well over 300 g l^{-1}(mummichog, *Fundulus heteroclitus*; Rogers, 1969).

Assuming that the species used in our experiments were representative of estuarine fishes, we have established a tentative classification of species types according to their capacity to survive in concentration of suspended solids. This classification was based primarily upon our work with fuller's earth, and uses as its foundation the LC 10 level (the calculated concentration at which 10% of the test animals would be expected to die in a 24-hour period). We have used the LC 10 as we consider a 10 percent induced mortality, in addition to natural mortality, as a more realistic allowable maximum than the more commonly used LC 50.

The more tolerant species observed were the mummichog (*F. heteroclitus*) spot (*Leiostomus xanthurus*) and the striped killifish (*Fundulus majalis*) (Table 4). From the results of Rogers' work, we classify the cunner (*Tautogolabrus adspersus*), the four-spined stickleback (*Apeltes quadracus*) and the sheepshead minnow (*Cyprinodon variegatus*) as "tolerant" species. In addition, we would classify those organisms in which we failed to induce mortality at concentrations of aobut 198 g l^{-1} as "tolerant". These included the oyster

Table 4. LC10, LC50 and LC90 values determined for 24-hr. exposure of estuarine fishes. Correlation coefficients (r) and coefficients of determination (r^2) derived from regression analyses are presented as statistical estimates of the decimal fraction of mortality accounted for by concentration effects.

Species	r	r^2	Lethal concentration, g l^{-1} Fuller's earth		
			LC10	LC50	LC90
White perch	0.94	0.880	3.05	9.85	31.81
Spot	1.00	1.000	13.08	20.34	31.62
Bay Anchovy	0.76	0.577	2.31	4.71	9.60
Atlantic Silverside	0.80	0.650	0.57	2.40	10.00
Mummichog	0.95	0.906	24.47	39.00	62.17
Striped killifish	0.97	0.934	23.77	38.19	61.36

toadfish (Opsanus tau), the hogchoker (Trinectes maculatus) and the cusk eel (Rissola marginata).

A common feature of the "tolerant" species is a habitat preference which tends toward the mud-water interface, a zone of the aquatic system where suspended solids concentrations tend to be higher than elsewhere in the water column. We propose that most species which carry out a majority of their life history associated with the mud-water interface would be relatively more tolerant of particles in suspension than other species.

Species found to be "sensitive" to the effects of suspended solids were the menhaden (B. tyrannus), the white perch (Morone Americana), and the bay anchovy (Anchoa mitchilli; Table 5). Common biological or ecological characteristics were difficult to ascertain for these species. Consideration of other forms (striped bass, M. saxatilis; croaker, Micropogon undulatus; and weakfish, Cynoscion regalis) which, although never studied in full, probably fall in the category of "sensitive" species, sheds some light on characteristics common to the sensitive forms.

Two of the species classified as "sensitive" were filter feeders (menhaden and anchovy) deriving their sustenance from plankton filtered from the water by the gills. This characteristic would lead one to infer a high potential sensitivity of filter feeders to suspended solids. The other species in the classification, however, are not filter feeders, and spend some substantial portion of their time feeding on and in the bottom sediments. White perch and striped bass, for instance, have similar food habits as juveniles, preying upon the benthos. Based on these considerations one would predict, as for spot, toadfish and the killifishes, a classification of "tolerant". Factors other than habitat preference and pre-adaptation to turbid environments must be operative, therefore, in the determination of a species' sensitivity to suspended solids. Studies of metabolism and respiratory physiology (see below) have demonstrated that at least some of these factors relate specifically to a fish's general level of biological activity and metabolism.

The Atlantic silverside was classified as a "highly sensitive" species (Table 4); juvenile and young-of-the-year life history stages of a number of species were also found to be classified as highly sensitive. These forms included juvenile bluefish (Pomatomus saltatrix) and young-of-the-year white perch (M. americana).

FACTORS IN SENSITIVITY

When fishes were exposed to lethal concentrations of fuller's earth, we observed that the gill filaments and the secondary lamellae acted as sieves to trap the sediment particles. Since the physical dimensions of the fish gill increase with increased size the size of the "filter" would also increase. Thus, the lethal effect of a given concentration of suspended solids would decrease for larger fish; fewer fine particles would be trapped by the gill.

Gill dimension is an interactive factor when considered in relation to different age classes of fish within a single species and their relative sensitivity to suspensions of mineral solids. The function of the gill is to provide, with a limited space, the maximum surface area for gas exchange and an optimum environment-to-organism gas transfer mechanism. This is accomplished by the fine folding of the gill surface into filaments, lamellae and secondary lamellae. At the secondary lamellae the gas transport tissue, blood, is separated from the environment by a double layer of cells, an epithelial layer, and an endothelial layer. The exchange of gases across the gill surface is such that, for given levels of metabolic demand, quantities of oxygen are removed from the water and quantities of carbon dioxide are excreted to the water by the fish. Smaller fishes, those with the finer gill filters, require more oxygen per unit of body weight than larger fish. Thus, the demand for a smaller individual to have the maximum surface area available for gas exchange and the greater potential for the gill of the smaller fish to clog in turbid conditions interact to make juveniles more sensitive than adults to suspended solids.

Exposure of fish to highly turbid waters generally resulted in violent displays of escape behavior: incessant "sidling" in the corners of the tanks; leaping from the water; "coughing" and rapid swimming into the walls of the tank. It is an understatement to say that all of these are manifestations of "stress".

We attempted to categorize the "stress" reaction in physio-logical terms using suspended sediment concentrations well below those necessary to induce mortality. In doing so we identified a "syndrome" of fish response to suspend solids, a syndrome common to fishes exposed to toxic materials in general.

Chronic, sublethal exposures of fish were carried out at between 0.45 and 0.65 g l^{-1} solids for periods of four to fourteen days. The general results were as follows:

Gill histology

Normal, or control, gills showed low concentrations of mucus goblet cells, those which normally produce mucus to lubricate and cleanse the gill. Experimental fish showed extremely heavy concentrations of mucus cells.

Hematology

Control fish demonstrated little or no change in several critical blood parameters in the course of four to 14 days in holding tanks (Table 5). Experimental fish demonstrated a general increase in specific parameters. These were; erythrocyte count, hemoglobin and hematocrit. Blood osmolality did not change, demonstrating that the observed responses were not due to osmotic responses of the fish to conditions during the test (Table 5).

The above responses are generally interpreted as an involuntary response on the part of the fish to the suspended solids by increasing the oxygen-carrying capacity of the blood. As shown previously, exposure of fish to suspended solids resulted in clogging of the gills. Exposure to sublethal concentrations of solids would be expected to result in only a partial clogging of the gill. With this clogging, the gas-exchange area of the gill would be reduced. Hematological parameters are plastic enough to compensate for this loss in gill surface area by increasing the concentration of red blood cells and hemoglobin factors active in the transport of respiratory gases.

Gill structure

Histological examination of the respiratory surfaces of the gills in test and control fish revealed a deterioration in the structure of the secondary lamellae and in the pilar cell (structural components) integrity in the lamellae.

Although gill damage due to suspended solids exposure has been demonstrated in our work and in studies by others (Herbert and Merkens, 1961) the damage has not been positively identified as being harmful to fish in terms of the overall survival rate. Many freshwater species (Ellis, 1937; EIFAC, 1964) have survived for several weeks under highly turbid conditions.

It is felt that compensatory reactions, such as those documented in our

Table 5. Control and experimental hematocrit values from the mummichog exposed for four, seven, and twelve days to a suspension of 1.62 g l^{-1} fuller's earth. Results expressed as mean $\pm$ s.d.

	4 Days	7 Days	12 Days
Experimental	33.08*	29.52**	34.14***
	$\pm$5.74	$\pm$4.43	$\pm$4.28
Control	24.14	23.79	26.52
	$\pm$6.54	$\pm$4.60	$\pm$2.24

*$p < 0.01$ ($t = 3.2488$, d.f. = 18).

**$p < 0.02$ ($t = 2.8373$, d.f. = 18).

***$p < 0.001$ ($t = 4.9884$, d.f. = 16).

studies of hematological responses to suspended solids, take over, enabling fish to survive despite the damage to the gill. In fact, shunt mechanisms may be employed by fishes under normal conditions so that not all the gill surface is in use (Randall, 1970). By employing the "reserve" surface area during prolonged exposure to suspended solids, fishes may possess sufficient functional gas-exchange surface to survive, and the functional decrease in surface area, should it occur, may be offset by compensatory increases in blood oxygen-carrying capacity.

We sought to learn more about the respiratory responses of fish to suspended solids through a series of experiments designed to determine the rates of oxygen consumption of fishes exposed to sub-lethal doses of fuller's earth and natural Patuxent River mud. Control fish gave the classical respiratory curve; a logarithmic increase in oxygen consumption with size of the fish tested. For fishes exposed to suspended solids, however, we observed a flattening of the curve in comparison to the controls. Overall, there was a reduction in respiration which varied, depending upon the species of fish tested (see e.g. Figure 2).

Finally, we observed a most interesting phenomenon when conducting autopsies: fishes exposed to lethal and sub-lethal concentrations of silt actually ingested the material. This phenomenon was primarily the result of mucus cleansing of the gill surface; the particle-laden mucus was observed streaming into the esophagus resulting in a substantial build-up of silt in the stomach and in the intestine. The presence of substantial quantities of particulate matter in the stomachs of experimental fish raised a most interesting question, that being, whether, in the course of dredging sediments having in them substantial quantities of toxins such as heavy metals, pesticide residues and the like, could result in accumulation of these toxins in the flesh of fishes. Suspended particle matter, particularly the finer particles of less than 2μ in diameter, often have toxins adsorbed to their surfaces and by exposure to the acid environment in a fish's stomach adsorbed materials may be released and become available to the fish. Studies have not yet been performed to determine whether this phenomenon occurs in fishes. In consideration of the quantities of contaminated sediments in U.S. harbors with which the Corps of Engineers must deal annually, the question deserves immediate attention.

CONCLUSIONS

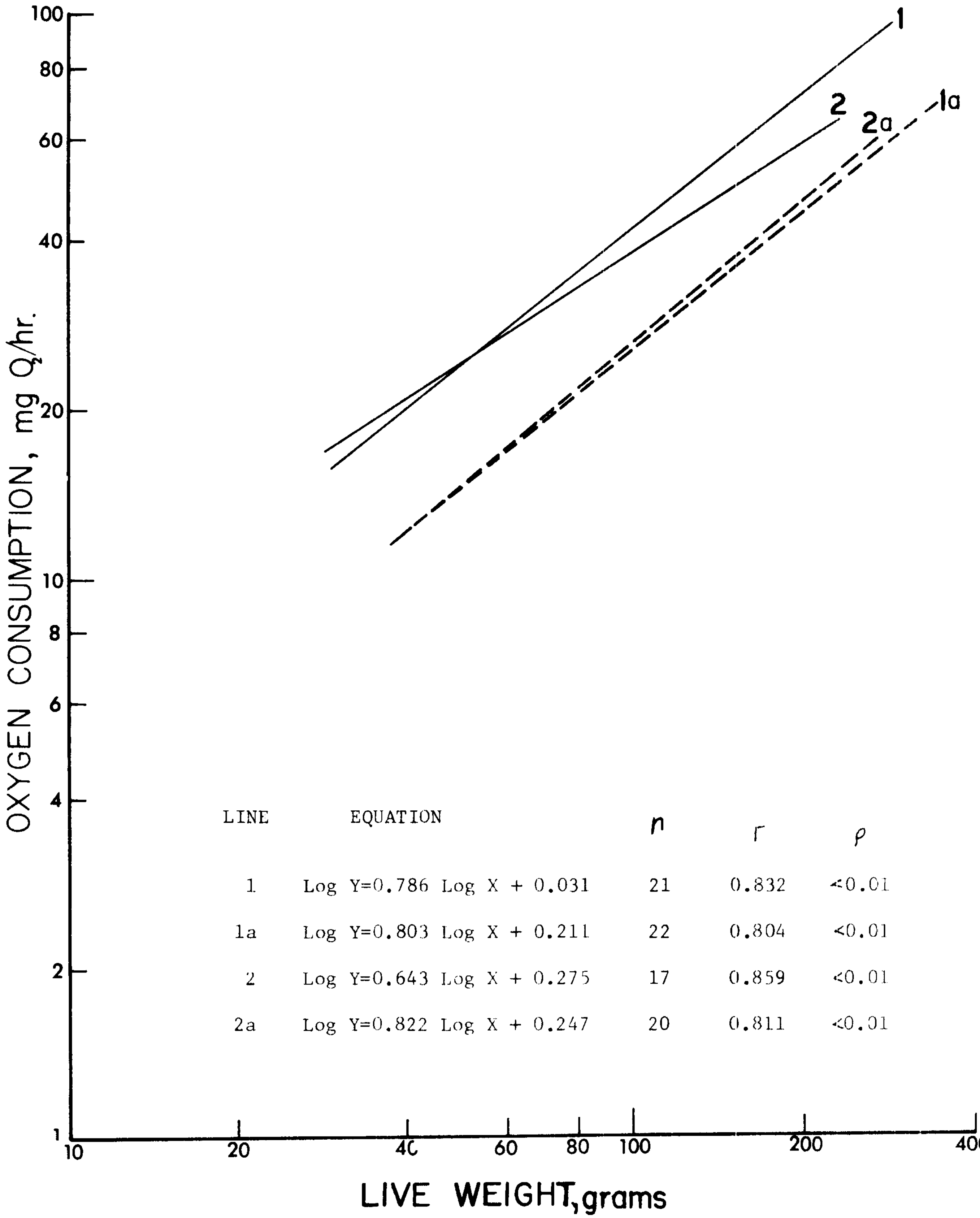
It is fair to state that our studies have generated more questions than actual answers or solutions to problems arising form heavy concentrations of suspended solids in estuarine environments. Many of these remaining questions were brought to the attention of the Waterways Experiment Station in 1970 and 1971 and have been included in the Corps of Engineers Dredged Materials Research Program.

Potential environmental hazards from dredging may be estimated accurately by knowing, prior to operation:

a. the particle size distribution of the material to be dredged and the volume of material to be removed

b. sufficient physical characteristics of the area to estimate accurately the settling time and aerial dilution of the finest particles to be resuspended

c. the biota of the affected area--phytoplankton, zooplankton, fish and benthos--and their ecology.



LINE	EQUATION	n	r	p
1	Log Y=0.786 Log X + 0.031	21	0.832	<0.01
1a	Log Y=0.803 Log X + 0.211	22	0.804	<0.01
2	Log Y=0.643 Log X + 0.275	17	0.859	<0.01
2a	Log Y=0.822 Log X + 0.247	20	0.811	<0.01

Figure 2.

Figure 2. Oxygen consumption of striped bass, M. saxatilis, at swimming speeds of 31.7 (1 and 1A) and 49.0 (2 and 2A) cm/sec at 22.5°C, Solid lines represent data for fish in filtered Patuxent River water; broken lines represent data for fish exposed to 1.30 (31.7 cm/sec) and 1.31 (49.0 cm/sec) g/liter Patuxent River sediment. Regression equations, number of fish tested (n), and correlation coefficients (4) with significance levels (P) are also shown.

Conclusions deriving from our studies may be summarized as follows:

- Suspended solids have significant effects on primary productivity, zooplankton feeding rates, fish survival and the physiological state of fishes. Although the experiments conducted used heavy concentrations of solids, the scope of responses observed was deemed meaningful as a general phenomenon, and may be more pronounced among less hardy species in natural environments.
- The lethal and sublethal effects of resuspended natural muds were less than those of mineral solids.
- The effects of suspended sediments on phytoplankton and zooplankton appeared to be related to particle concentration rather than to particle size. The results obtained from test with phytoplankton were basically those one would expect from reduction in light intensity.
- Dredging and spoiling operations are capable of producing suspended solids concentrations capable of affecting the natural function of organisms and ecosystems as a whole. It is unlikely that harmful concentrations of suspended solids would persist for a period of time sufficient to induce meaningful lethal or sublethal effects on most communities.

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